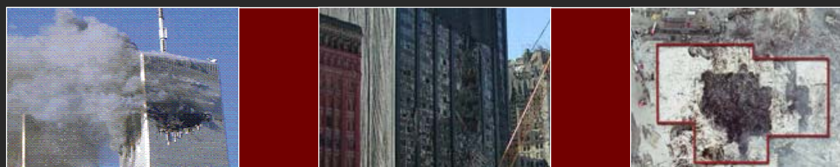

Ronald O. Hamburger, SE

Alternative Methods of Evaluating and Achieving Progressive Collapse Resistance

National Steel Construction Conference

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Simpson Gumpertz & Heger Inc.
Consulting Engineers
Boston / San Francisco / Washington DC

Acknowledgement

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- Advanced Technology Institute
- Corps of Engineers,
  Civil Engineering Research Laboratory

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  State University of NY at Buffalo
- Mr. Donald Dusenberry, PE
  Simpson Gumpertz & Heger, Inc.

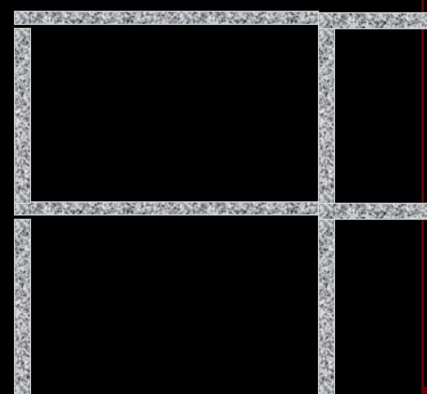
What is Progressive Collapse?

- A small, localized initiating event
- leads to a chain of failures
- resulting in large scale failure and collapse

- Since nearly all collapse is progressive in nature
- Any structure will collapse if subjected to a sufficiently large event
- Design goal is to prevent small events from initiating large scale failures
  - “disproportionate failure”

Ronan Point - 1968

- 23 story apartment building in London
- Precast concrete panel construction



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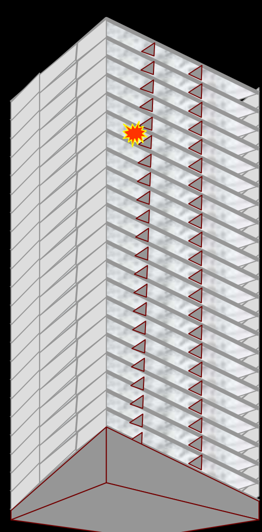
Ronan Point - 1968



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Alfred P. Murrah Building

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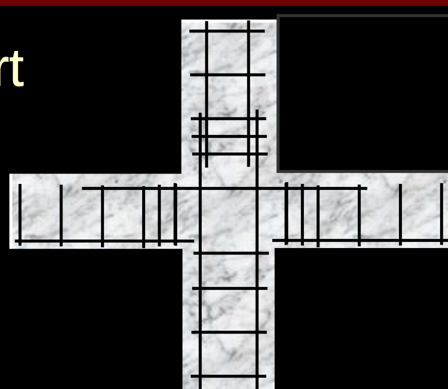
ASCE Report



- Incorporation of seismic detailing would have provided the Murrah Building with greater resistance to the progression of collapse
  - Continuity of reinforcing
  - Confinement of members for ductility

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Steel Moment Frames are an Excellent System for Arresting Collapse

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Steel Moment Frames are an Excellent System for Arresting Collapse

Steel Moment Frames are an Excellent System for Arresting Collapse

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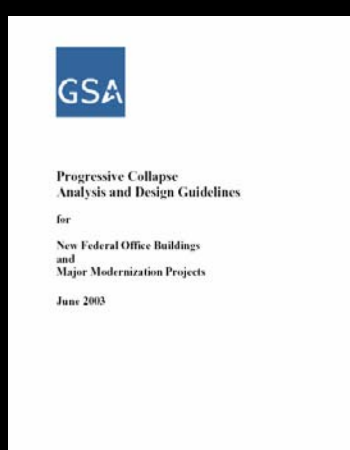
Design Criteria for Progressive Collapse



Progressive Collapse
Analysis and Design Guidelines

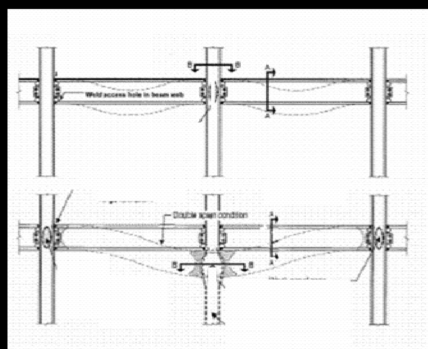
for

New Federal Office Buildings
and
Major Modernization Projects

June 2003

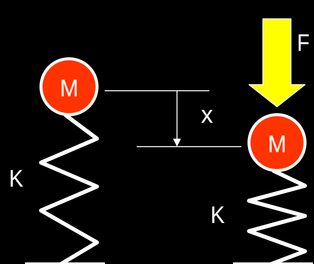
- Provide frame redundancy & continuity
- Evaluate adequacy of continuous framing for column removal condition
  - Linear static procedure
  - Nonlinear static procedure
  - Nonlinear dynamic procedure

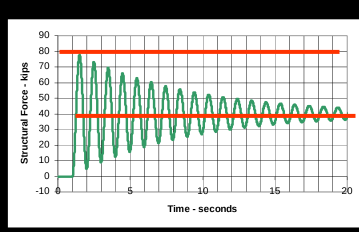
Linear Static Procedure



- Structure evaluated for element removal
  - Demand taken as:
    - 2(D+0.25L)
  - Capacity taken as ZF_{ye}
  - Demand/Capacity Ratios evaluated against acceptable values from seismic guidelines (FEMA 356)
    - Compact sections – 3
    - Noncompact sections - 2

Basis





- Structure evaluated for element removed
  - Demand taken as:
    - 2(D+0.25L)
  - Basis:
    - Expected load actually present is dead load D and 25% of live load L
    - Peak deflection and stress on an elastic, lightly damped structure is twice the static value
    - Presumption is made that this is a conservative bound on collapse behavior

Basis

Component/Action	Values for Linear Procedures DCR
Beams – flexure	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	3 3
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	2 2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.
Columns – flexure	
For $0 < P/P_{CL} < 0.5$	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	2
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	1.25
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.

- Demand – capacity ratios obtained from FEMA 356
  - Intended to represent modestly conservative estimates of ductility available in elements under seismic loading
  - Presumption is made that similar ductility is available under collapse conditions

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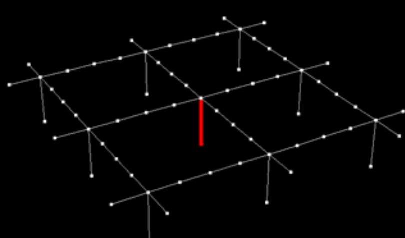
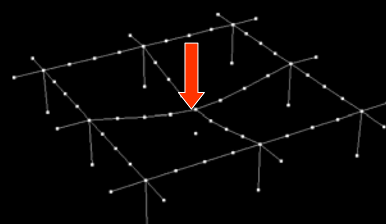
Nonlinear Static Procedure





- Structure modeled with removed element "missing"
- Gravity load pattern applied and scaled to produce first yield
- Model modified to reflect yield hinge
- Load incremented until next hinge predicted
- Process repeated until mechanism forms or 2(D+.25L) resisted
- Maximum ductility demands evaluated against seismic capacities
  - Plastic rotations comparable to linear acceptance criteria

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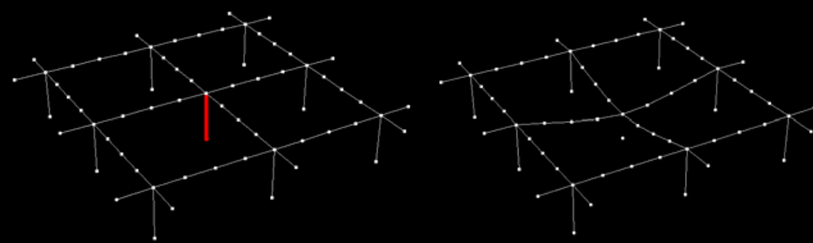
Nonlinear Dynamic Procedure



- Structure modeled
- Element removed
- Nonlinear time history analysis performed to determine response
- Maximum ductility demands evaluated against seismic capacities
  - Catenary tensile behaviors can be accounted for
  - Plastic rotations comparable to linear acceptance criteria

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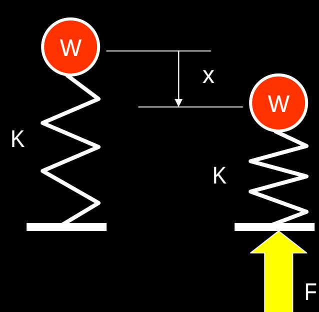
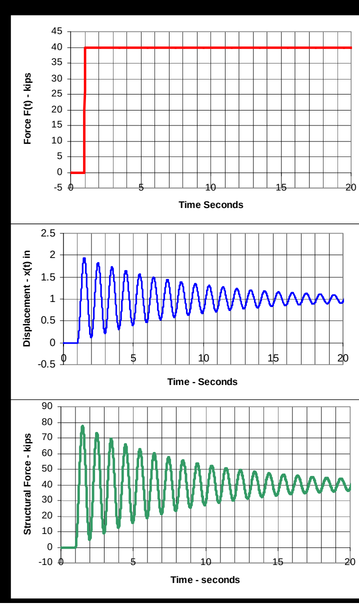
Problems with linear method

- “Impact” load increase value of 2 is based on elastic behavior



Linear Behavior
W=40k F_{max}=80k



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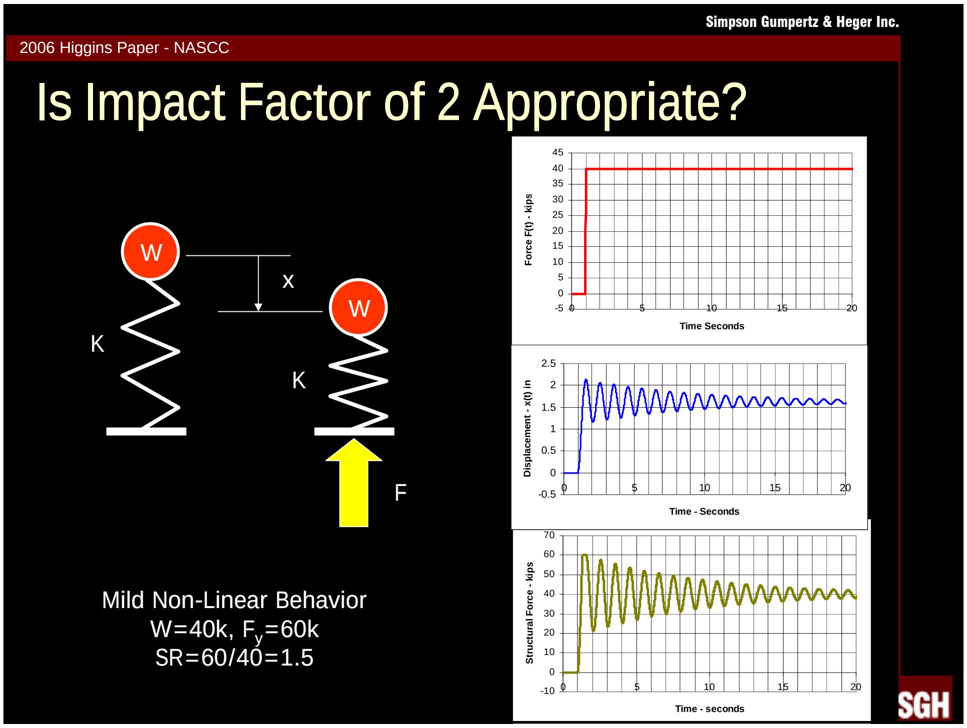

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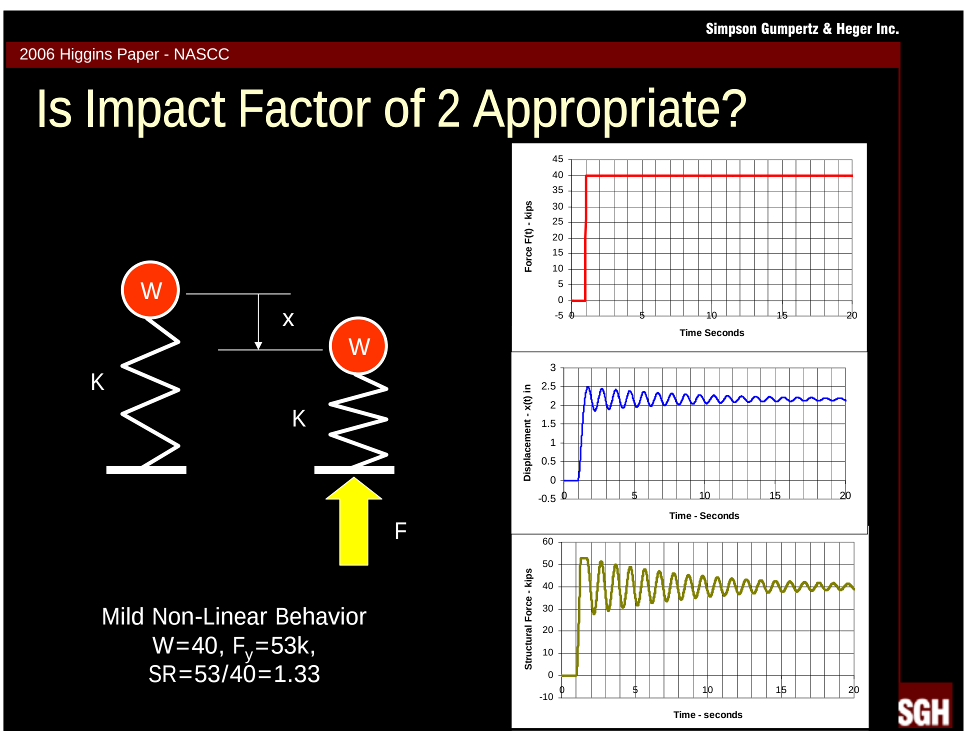
Is Impact Factor of 2 Appropriate?
W
x
W
K
K
F
Mild Non-Linear Behavior
W=40k, Fy=60k
SR=60/40=1.5
Force F(t) - kips
Time Seconds
Displacement - x(t) in
Time - Seconds
Structural Force - kips
Time - seconds
SGH



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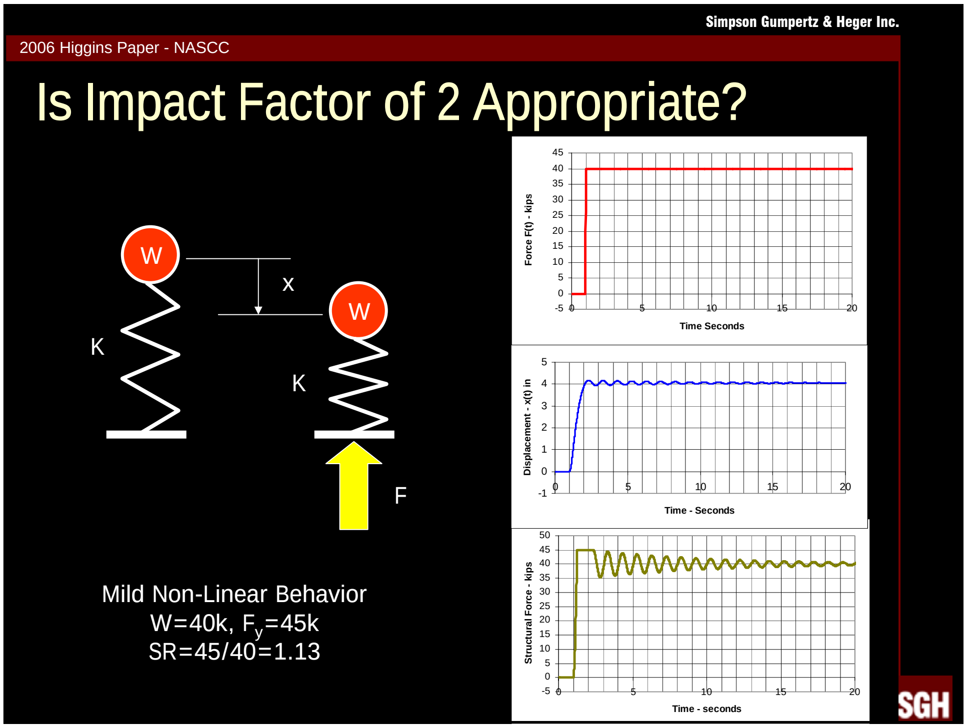
Is Impact Factor of 2 Appropriate?
W
x
W
K
K
F
Mild Non-Linear Behavior
W=40, Fy=53k,
SR=53/40=1.33
Force F(t) - kips
Time Seconds
Displacement - x(t) in
Time - Seconds
Structural Force - kips
Time - seconds
SGH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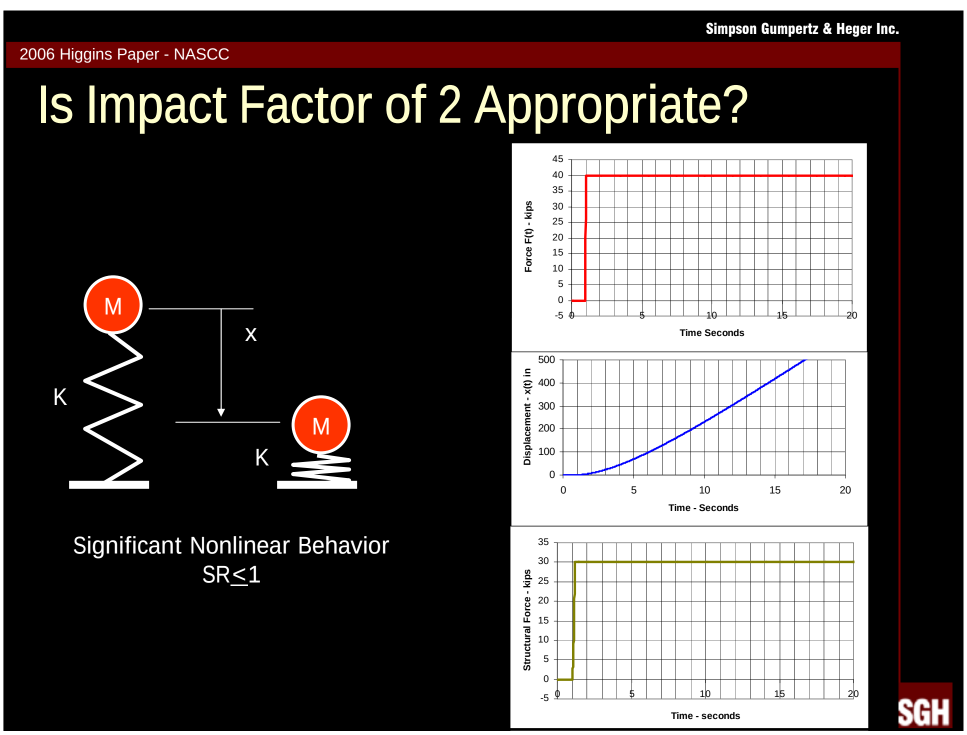
Is Impact Factor of 2 Appropriate?
W
x
W
K
K
F
Mild Non-Linear Behavior
W=40k, F_y=45k
SR=45/40=1.13
Force F(t) - kips
Time Seconds
Displacement - x(t) in
Time - Seconds
Structural Force - kips
Time - seconds
SGH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Is Impact Factor of 2 Appropriate?
M
x
M
K
K
Significant Nonlinear Behavior
SR≤1
Force F(t) - kips
Time Seconds
Displacement - x(t) in
Time - Seconds
Structural Force - kips
Time - seconds
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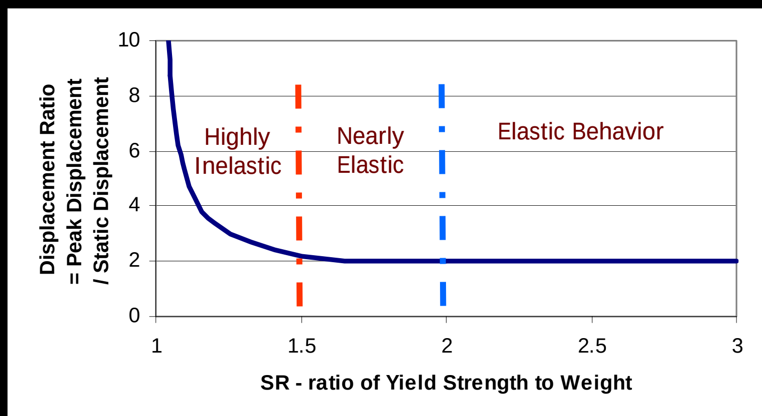
Displacement & Strength Ratios



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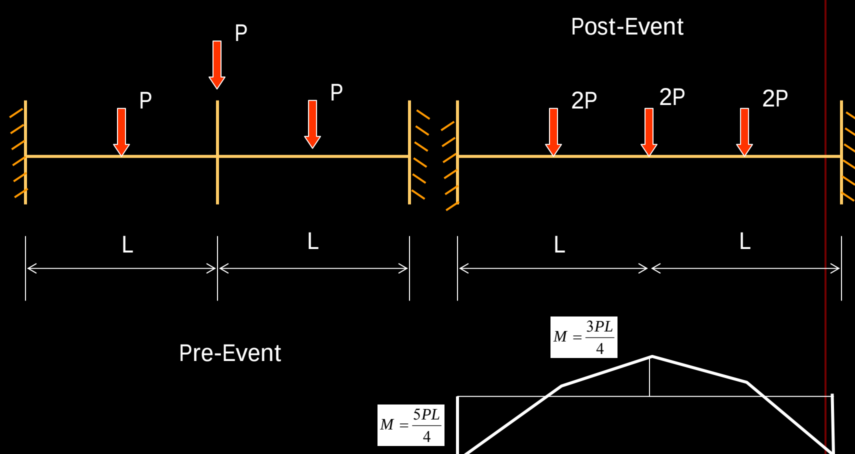
Linear Procedures Can Result in Structures that are too weak



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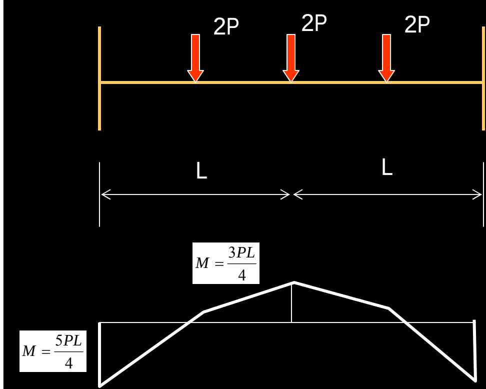
Linear Procedures Can Result in Structures that are too weak

Post-Event



$$\text{Permissible Minimum strength} = \frac{M_{\max}}{DCR}$$

$$\text{Permissible DCR} = 2$$

$$M_{p\min} = \frac{\left(\frac{5PL}{4}\right)}{2} = \frac{5PL}{8}$$

Linear procedure would permit use of structure to support gravity load P=8Mp/5L

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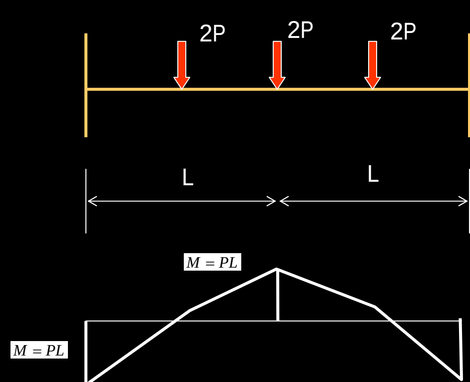
Linear Procedures Can Result in Structures that are too weak



Fully plastic condition achieved at P=Mp/L

Value of P=8Mp/5L has strength ratio of 5/8 and would collapse

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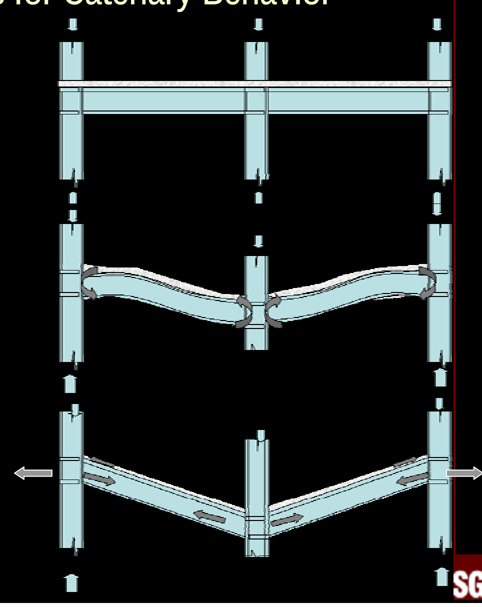
Nonlinear static procedure is also incorrect

- Impact factor of 2 is excessively conservative when nonlinear behavior is directly accounted for

Neither Procedure Accounts for Catenary Behavior

- Following development of full plastic flexural capacity of framing, beams fall into large deflection and catenary tensile behavior may develop, greatly increasing plastic capacity



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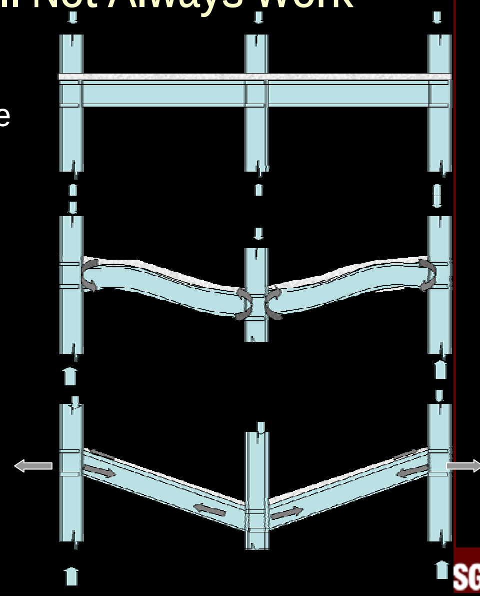
Catenary Behavior Will Not Always Work

- Can not develop unless structure can resist large tensile forces that develop at edge of collapsed area
- Behavior is not predictable by linear methods of analysis
- Catenary behavior typically can not be developed effectively at building edges and corners



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Energy-based Pushdown Analysis Procedure

- Adapted from pushover procedures currently used in earthquake engineering
- Can be performed as a series of sequential linear analyses
- Cannot account for catenary behavior
- Can more realistically model plastic flexural behavior of structure and ability to arrest collapse than linear methods

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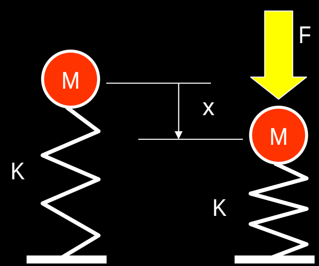
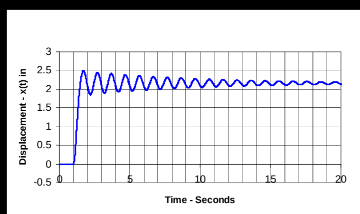
Base Theory





- As a mass supported by a spring (or damaged structure) drops under the action of gravity forces:
  - Mass loses potential energy (PE)
  - Mass gains kinetic energy (KE)
  - Spring gains strain energy (SE)
- At any instant of time (t)
  - KE = PE-SE
- As long as kinetic energy continues to increase, mass continues to fall
- If kinetic energy becomes negative (stored strain energy exceeds loss in potential energy) this indicates mass has reversed direction of travel and begun safe oscillation

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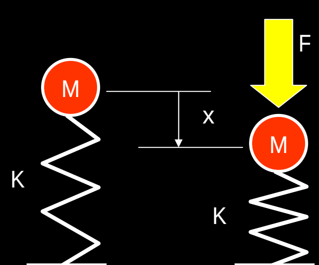
Base Theory



- At displaced position, x the potential energy lost by the falling mass is given by:

$$PE = Wx = Mgx$$

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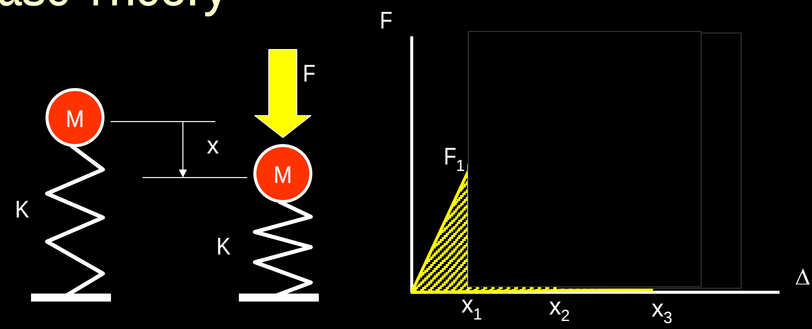
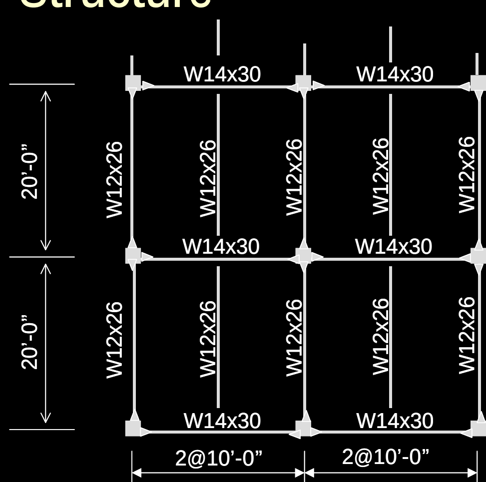
Base Theory

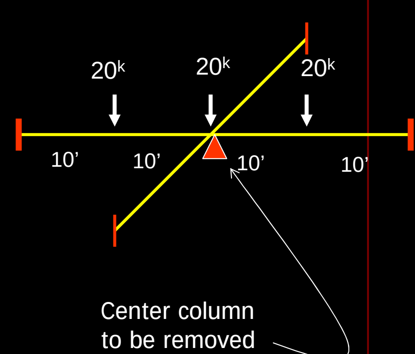


- At any displaced position x_i, strain energy is given by the area under the force – displacement curve

$$SE_1 = \frac{F_1 x_1}{2} \qquad SE_2 = SE_1 + \frac{1}{2}(F_1 + F_2)(x_2 - x_1) \qquad SE_3 = SE_2 + \frac{1}{2}(F_2 + F_3)(x_3 - x_2)$$

Illustration of Procedure Using Simple Structure





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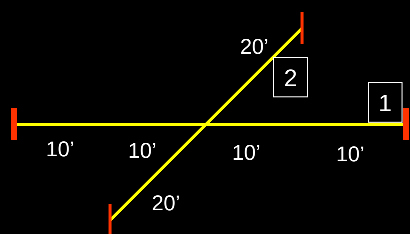
Procedure

1. Develop elastic model of structure, with “damaged” element removed



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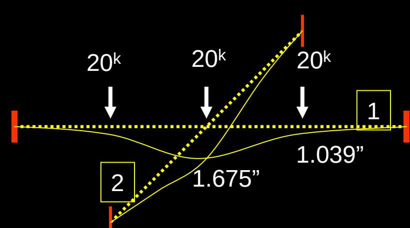
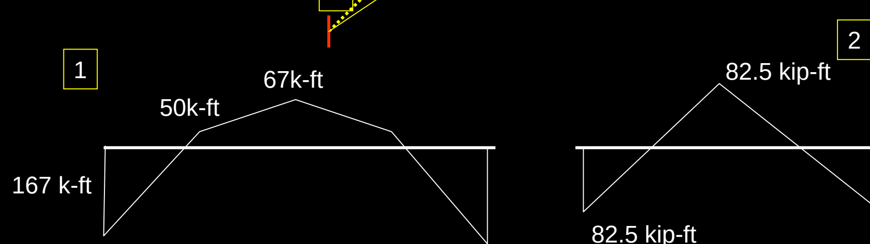
2. Apply weight W (D + .25 L) to model, calculate forces in all the elements and deflections at all loads





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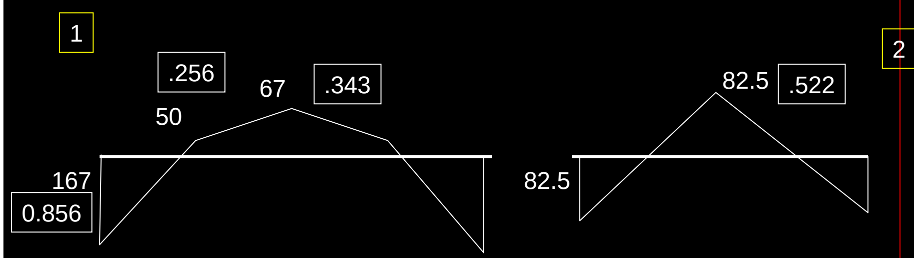
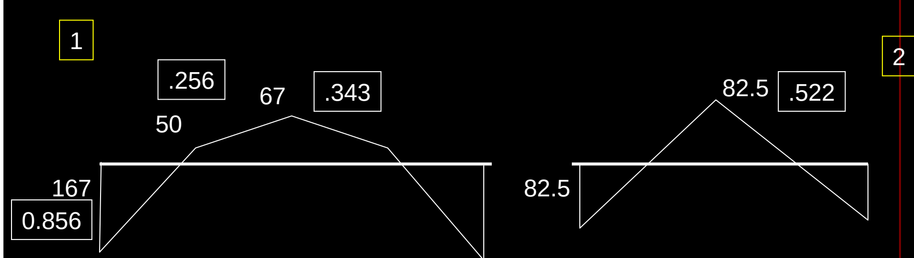
3. Calculate Demand to Capacity Ratios (DCRs) for all elements

W14x30 M_p=195 kip-ft, W12x26 M_p=158 kip-ft



4. Find the factor on the applied static loading W that causes first yielding of structure as $\beta_1 = 1/DCR_{max}$



$\beta_1 = 1/0.856 = 1.168$

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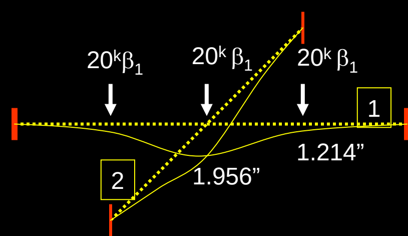
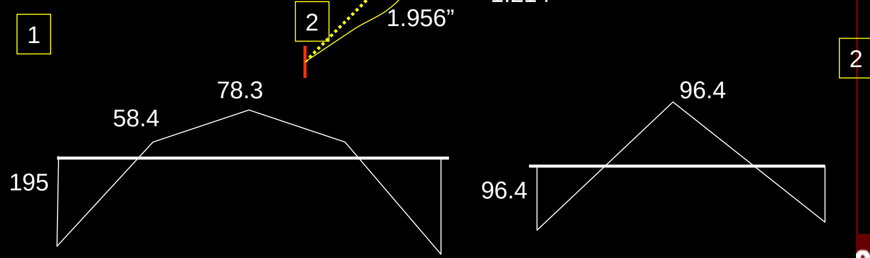
5. Find the forces in all elements, and the deflections under all loads at applied load $\beta_1 W$





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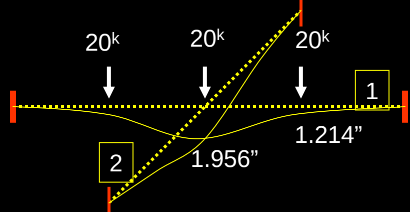
6. Calculate the total loss in Potential Energy as the structure deflects to a position β_1 times the static deflection under weight W

$$PE_1 = \sum_{i=1}^{n} W_i \Delta_i$$



Node "I"	Wi	Δi	WiΔi	
1	20	1.214	24.28	kip-in
2	20	1.956	39.12	kip-in
3	20	1.214	24.28	kip-in
Total			87.68	kip-in

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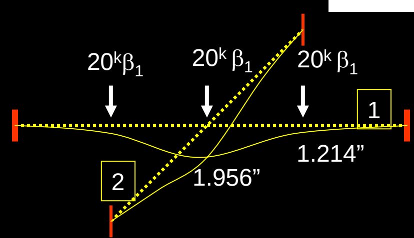
7. Calculate the total gain in Strain Energy as the structure deflects to a position β_1 times the static deflection under weight W

$$SE_1 = \frac{1}{2}\sum_{i=1}^{n} \beta_1 W_i \Delta_i$$



Node "I"	β_1Wi	Δi	0.5WiΔi	
1	23.36	1.214	14.18	kip-in
2	23.36	1.956	22.85	kip-in
3	23.36	1.214	14.18	kip-in
Total			51.21	kip-in

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8. Perform energy balance to determine Kinetic Energy

$$KE_1 = PE_1 - SE_1$$
$$KE_1 = 87.68 - 51.21$$
$$KE_1 = 36.47 kip - in$$

Conclusion: Kinetic energy is positive, structure has not been able to arrest collapse at deflection of β_1 times static deflection of structure under weight

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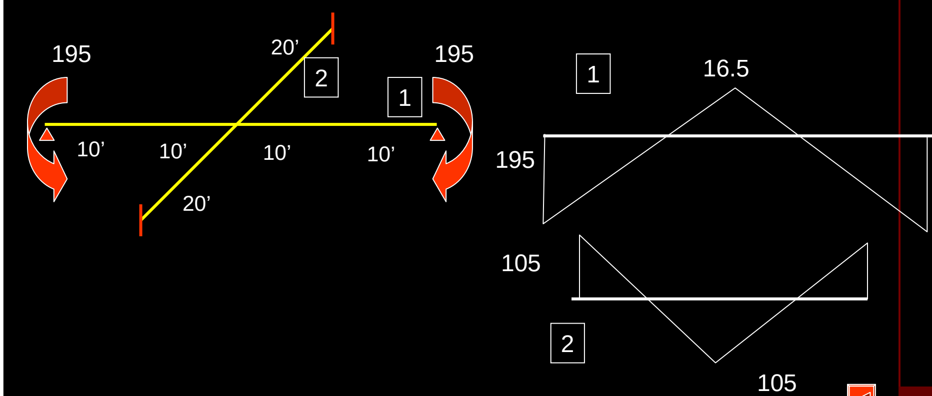
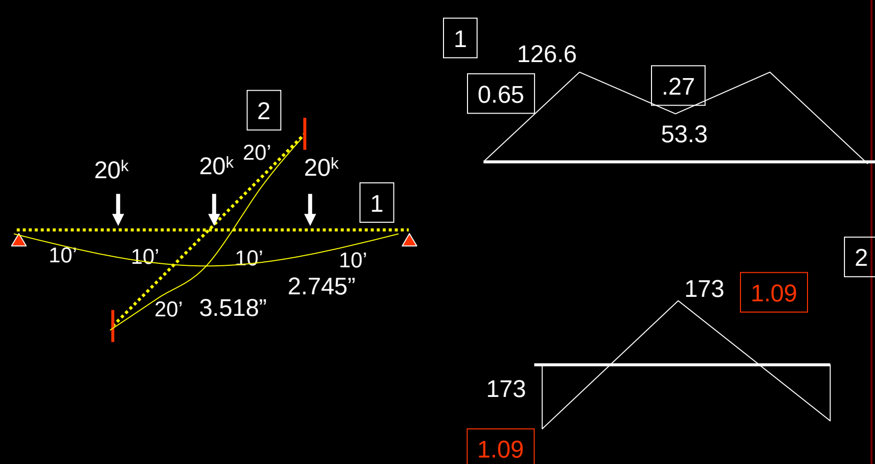
9. Place releases in model at point(s) where yielding occurs under $\beta_1 W$ forces. Apply plastic capacity of structure at this point(s) to represent yielding behavior of structure. Calculate forces, F_{1i}.



10. Calculate forces and DCRs in "released" model with applied load W



11. Find the factor on the applied static loading, W, that produces the next yield event in the structure, β_2.

Where:

$$\beta_2 = \frac{1 - \frac{F_{1i}}{M_{pi}}}{DCR_{2i}} = \frac{1 - \frac{-105}{158}}{1.09} = 1.527$$

$DCR_{2i\,max}$	Is the maximum DCR in the "released" structure under applied loading W (1.09)
"i"	Is the element degree of freedom at which this maximum DCR occurs
M_{pi}	Is the plastic capacity of the structure at the degree of freedom with the maximum DCR (158 kip-ft)
F_{1i}	Is the force at the degree of freedom resulting from the application of prior release forces (105 kip-ft)

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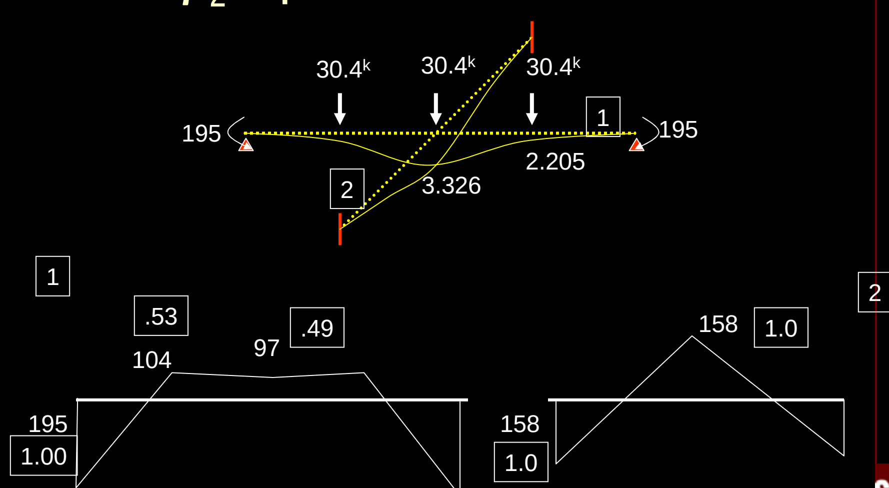
12. Evaluate structure with releases for β_2W plus release forces



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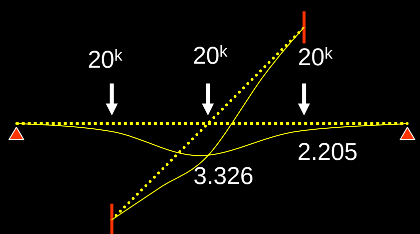
13. Calculate potential energy lost at this applied deflection

$$PE_2 = \sum_{i=1}^{n} W_i \Delta_i$$



Node "I"	Wi	Δi	WiΔi	
1	20	2.205	44.1	kip-in
2	20	3.326	66.52	kip-in
3	20	2.205	44.1	kip-in
Total			154.72	kip-in

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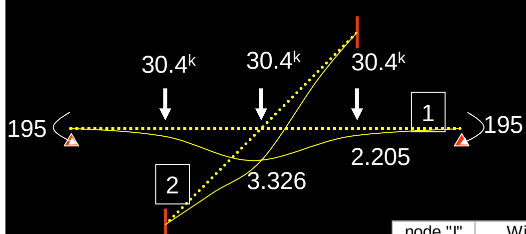
14. Calculate Strain Energy gained at this applied deflection



$$SE_2 = SE_1 + \frac{\beta_1 + \beta_2}{2} \sum_{i=1}^{n} W_i (x_2 - x_1)$$

node "I"	Wi	Δ2i	Δ1i	Wi(Δ2i-Δ1i)	
1	20	2.205	1.214	19.82	kip-in
2	20	3.326	1.956	27.4	kip-in
3	20	2.205	1.214	19.82	kip-n
Total				67.04	kip-in
	β1	1.168			
	β2	1.527			
SE1	51.21	kip-in			
SE2	141.55	kip-in			

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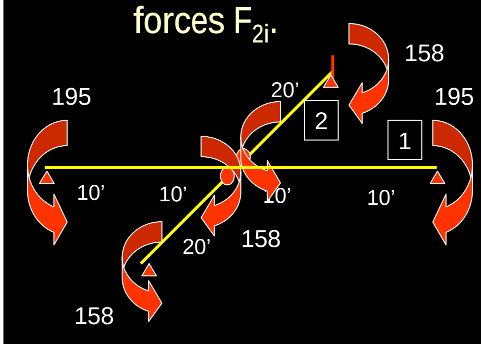
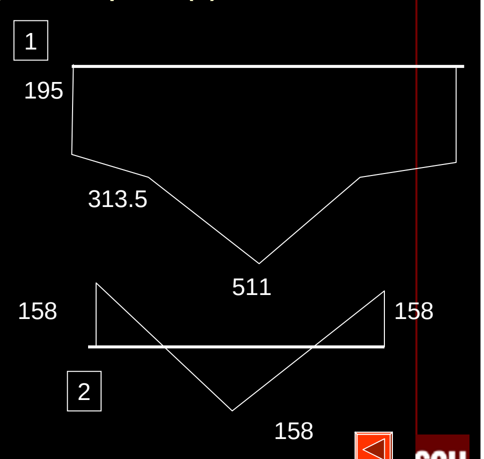
15. Compute Kinetic Energy at this displacement

$$KE_2 = PE_2 - SE_2$$
$$KE_2 = 154.72 - 141.55$$
$$KE_2 = 13.17 kip - in$$

Conclusion: Kinetic energy is still positive, structure has not been able to arrest collapse at deflection of β_1 times static deflection of structure under weight

16. Place releases in model at point(s) where yielding occurs under β_2W deflections. Apply plastic capacity of structure at yielded point(s). Calculate forces F_{2i}.





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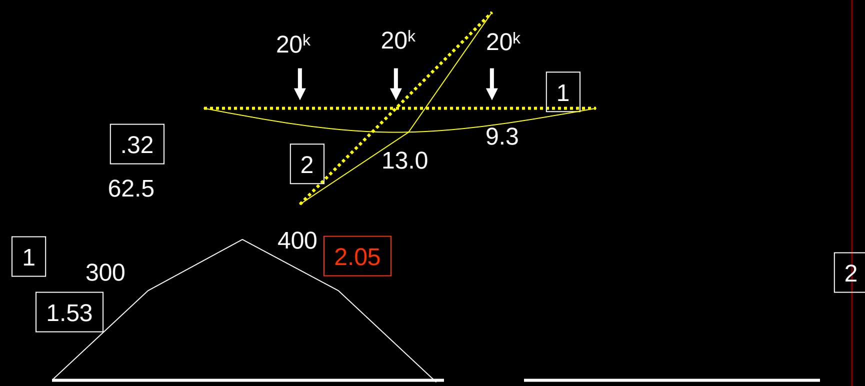
17. Apply weight W to structure with all releases and calculate DCRs



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18. Find the fraction of the deflection under static loading, W, that produces the next yield event in the structure, β_3.

Where:

$$\beta_3 = \frac{1 - \frac{F_{2i}}{M_{pi}}}{DCR_{3i}} = \frac{1 - \frac{-511}{195}}{2.05} = 1.77$$

$DCR_{3i\max}$ Is the maximum DCR in the “released” structure under applied loading W (2.05)

“i” Is the element degree of freedom at which this maximum DCR occurs

M_{pi} Is the plastic capacity of the structure at the degree of freedom with the maximum DCR (195 kip-ft)

F_{2i} Is the force at the degree of freedom resulting from the application of prior release forces (511 kip-ft)

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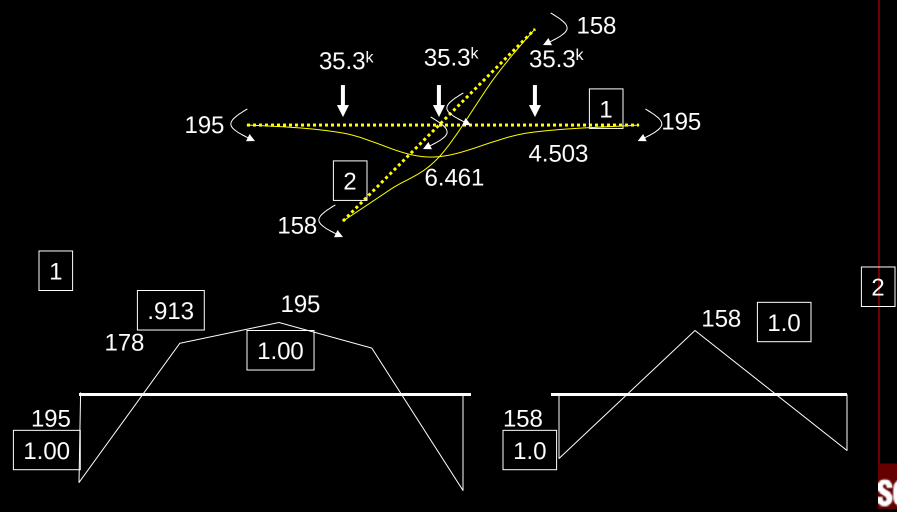
19. Evaluate structure for β_3 W plus release forces, simultaneously applied



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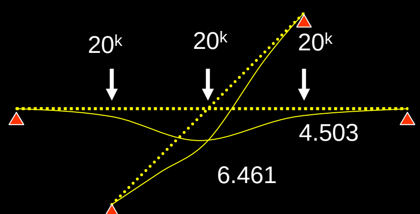
20. Calculate potential energy lost at this applied deflection

$$PE_3 = \sum_{i=1}^{n} W_i \Delta_i$$



Node "I"	Wi	∆i	Wi∆i	
1	20	4.503	90.06	kip-in
2	20	6.461	129.22	kip-in
3	20	4.503	90.06	kip-in
Total			309.34	kip-in

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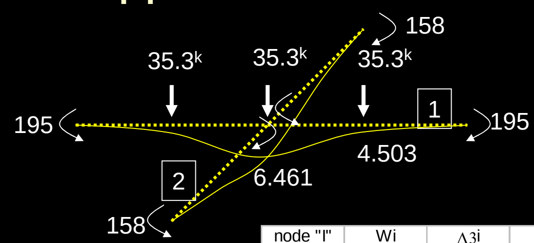
21. Calculate Strain Energy gained at this applied deflection



$$SE_3 = SE_2 + \frac{\beta_2 + \beta_3}{2} \sum_{i=1}^{n} W_i (x_3 - x_2)$$

node "I"	Wi	Δ3i	Δ2i	Wi(Δ3i-Δ2i)	
1	20	4.503	2.205	45.96	kip-in
2	20	6.461	3.326	62.7	kip-in
3	20	4.503	2.205	45.96	kip-n
Total				154.62	kip-in
	β2	1.527			
	β3	1.77			
SE2	141.55	kip-in			
SE3	396.44	kip-in			

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22. Compute Kinetic Energy at this displacement

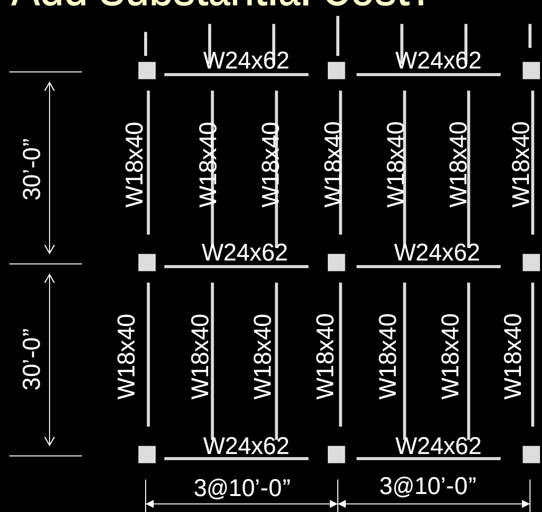
$$KE_3 = PE_3 - SE_3$$
$$KE_2 = 309.34 - 396.44$$
$$KE_2 = -87.1 kip - in$$

Conclusion: Kinetic energy is now negative, structure has been able to arrest collapse at a deflection less than β_3 times static deflection under the applied weight and initiate safe oscillation and damping out

Summary

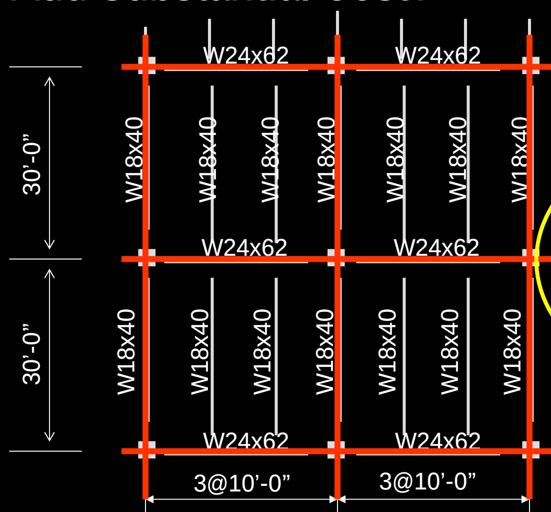
- The same types of pushover techniques used in performance-based earthquake engineering can be used to more accurately characterize the risk of progressive collapse than standard linear procedures
- The procedure does not require sophisticated software
- It can not account for catenary action – nonlinear dynamic analysis is required to evaluate this effect

Does Design for Collapse Avoidance Add Substantial Cost?



- 50 psf live load
- 20 psf partition
- 5 psf mech & electrical
- 3" 20 gauge deck w/ 2-1/2" lightweight conc
- Steel designed non-composite, using LRFD
- 6 psf floor framing

Does Design for Collapse Avoidance Add Substantial Cost?



- Moment-connect all beams and girders on column lines
- No change in member sizes
- Applying "pushdown" procedure – structure comes to rest after formation of hinges at girder ends, but before formation of any other hinges
- Conclusion – collapse avoidance can be achieved without weight increase but with significant additional connection costs

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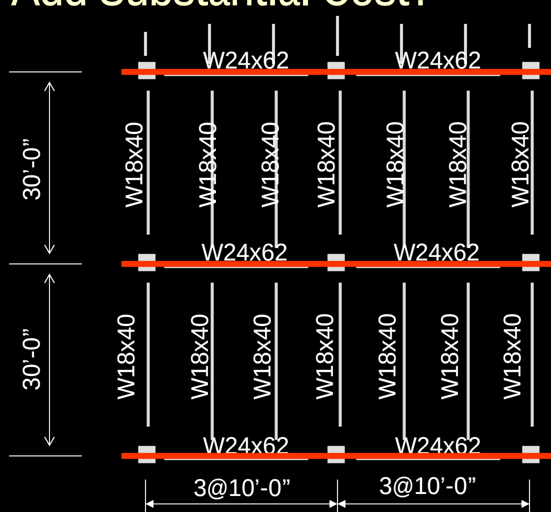
Does Design for Collapse Avoidance Add Substantial Cost?



- By observation:
  - Moment-connect girders only
  - No change in member sizes
- Conclusion – collapse avoidance can be achieved without weight increase but with some additional connection costs

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The Cost of Collapse Avoidance

- Steel weight 6 pounds/ft^2 (flat)
  3 pounds/ft^2 (columns)
  - Estimated cost –framing ($2,500/ton) $11.25/ft^2
  - Estimated cost of moment connections ($500 connection) $1.11/ft^2
  - Total Steel Cost $12.36/ft^2
- Premium for collapse prevention 10%
- Some significant schedule impact

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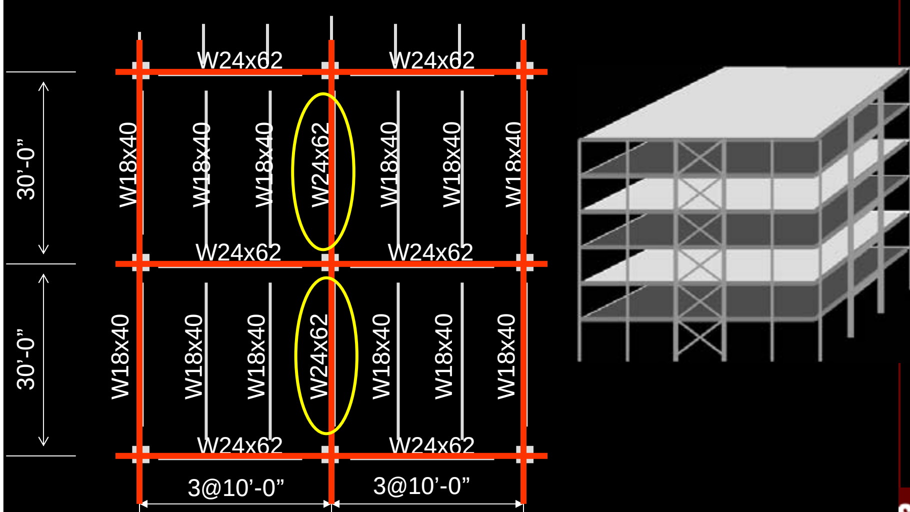
Can these impacts be reduced?



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The Cost of Collapse Avoidance

- Steel weight — 6.4 pounds/ft^2 (flat), 3 pounds/ft^2 (columns)
  - Estimated cost –framing $3,000/ton) — $11.75/ft^2
  - Estimated cost of moment connections ($500 connection) — $1.11/ft^2
  - Total steel cost - — $12.86/ft^2
- Premium for collapse prevention — 14%
- Same schedule impact

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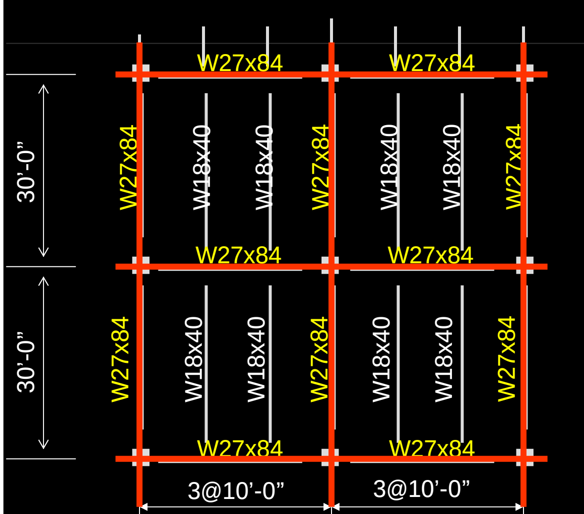
Can impacts be reduced?



- What change to framing is required to allow moment connections to be made only on each third floor?
  - Girders and beams must change to W27x84 on strong floor only

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The Cost of Collapse Avoidance

- Steel weight 6.8 pounds/ft^2 (flat)
  3 pounds/ft^2 (columns)
  - Estimated cost –framing $2,500/ton) $12.25/ft^2
  - Estimated cost of moment connections ($500 connection) $0.75/ft^2
  - Total steel cost - $13.00/ft^2
- Premium for collapse prevention 15%
- Schedule impacts reduced (fewer moment connectinos)

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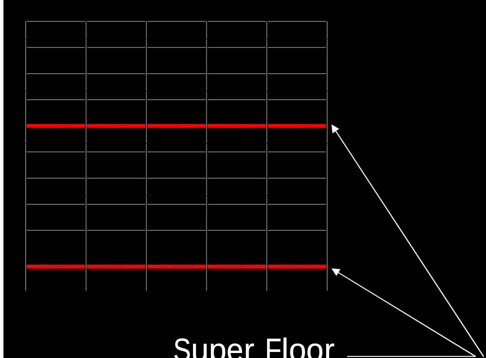
How many floors can be supported by one floor?



- Limits are:
  - Ability of connections to resist inelastic straining
  - Tolerance to deflection of floors in collapse avoidance condition

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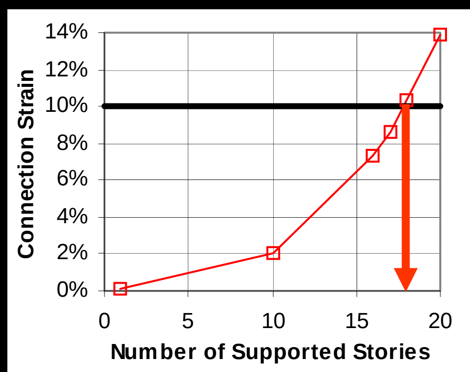
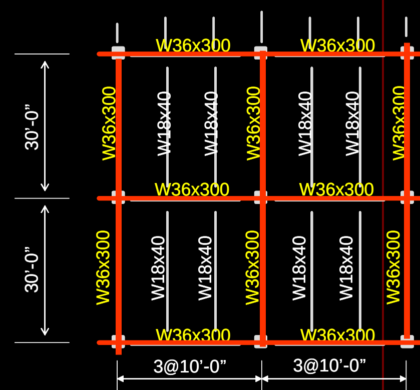
How many floors can be supported by one floor?





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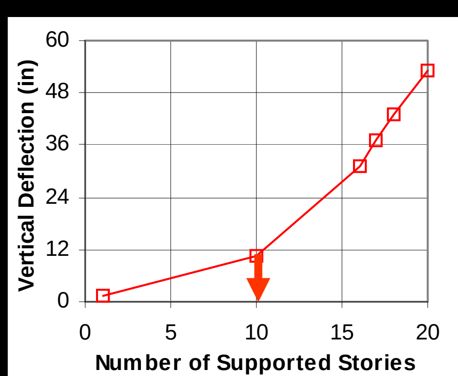

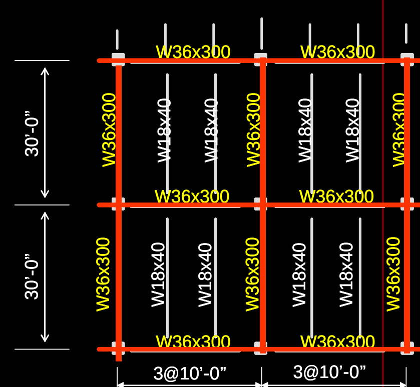

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Summary

- The “strong floor concept” can be used to economically provide collapse resistant structures without substantial cost or schedule premium
- The Devil is in the Details (connections)
- As an industry, we still do not have good understanding of the vulnerability of various types of connections to impact induced fracture

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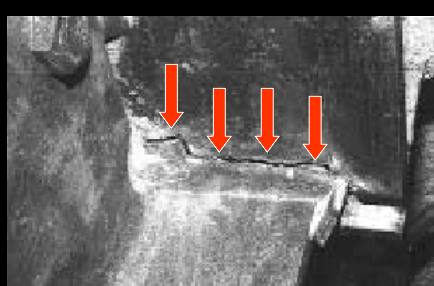
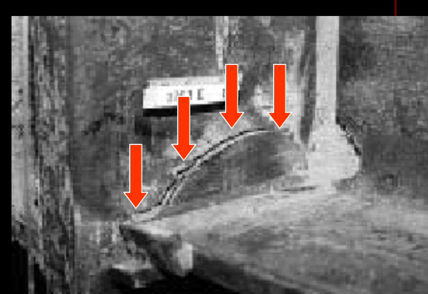
Connections are the critical issue





Welded, bolted flange moment-resisting connections fractured in the 1994 Northridge earthquake at modest demands

- yield level stresses
- negligible tension

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Similar and more severe damage has occurred in collapse conditions

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Similar and more severe damage has occurred in collapse conditions

Bolted Connections are also vulnerable

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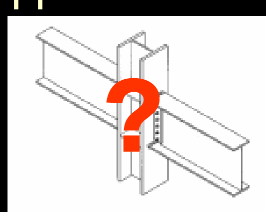
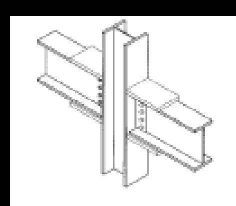
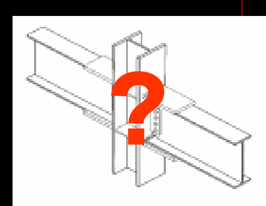
Guidelines permit use of a wide range of "approved connections"



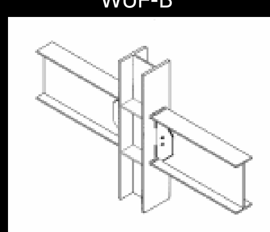
WUF-B



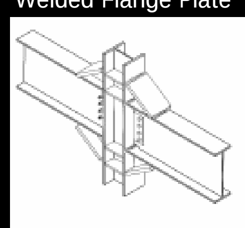
Welded Flange Plate



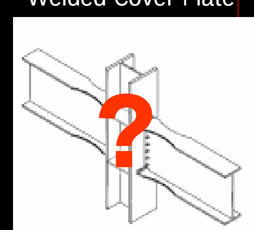
Welded Cover Plate



Free Flange



Haunched



RBS

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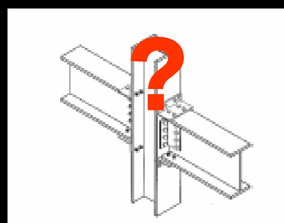
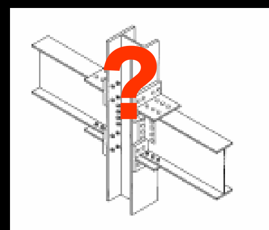
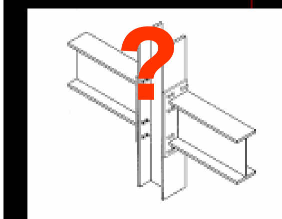
Guidelines permit use of a wide range of “approved connections”



Bolted Angle



Double Tee



End Plate

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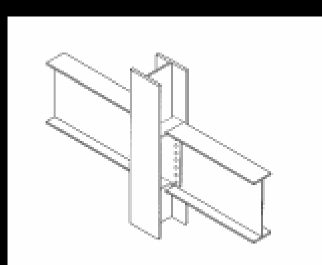
Moment-connections may not be necessary



- Catenary behavior is an effective method of resisting collapse
- Moment development at girder ends is not required for this
- Robust tensile connections may be sufficient to provide collapse avoidance capability

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Conclusions

- Steel framing has excellent ability to provide collapse resistance in structures
- The most common method of analysis used today is not adequate, but simple design approaches can be used to evaluate collapse resistance
- Collapse resistance can be provided with negligible increase in steel tonnage, cost or schedule.
- Connections are key
  - Ability of frame to provide collapse resistance
  - Relative cost of collapse resistance
- Testing of various connection technologies to demonstrate adequate robustness is urgently needed