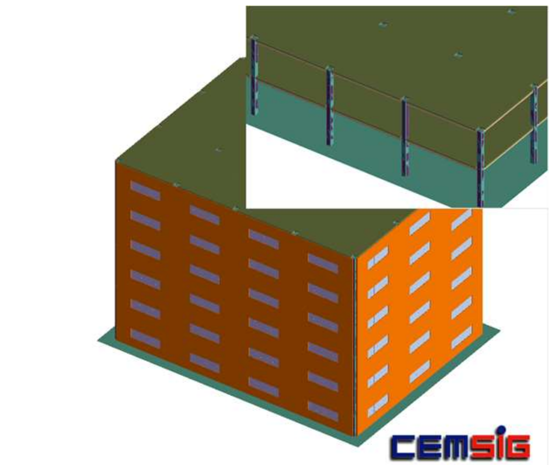
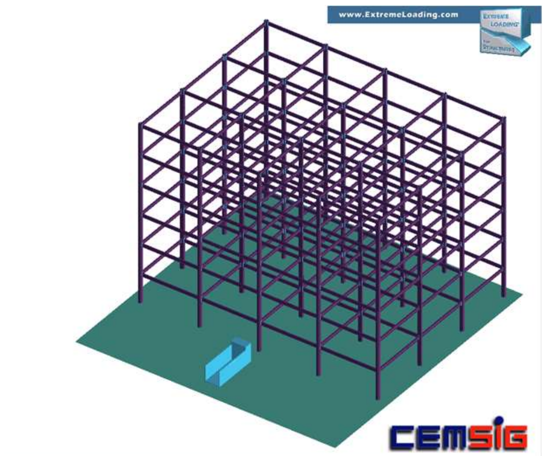
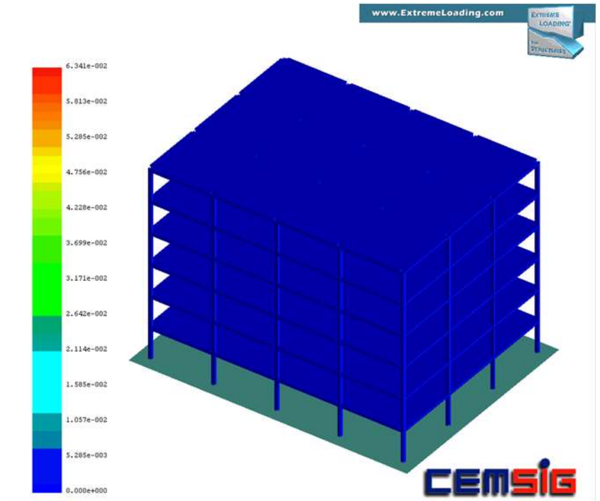
Robustness of structures







ADS Master course

Florea Dinu, Prof. PhD

2018-2019

Motivation:

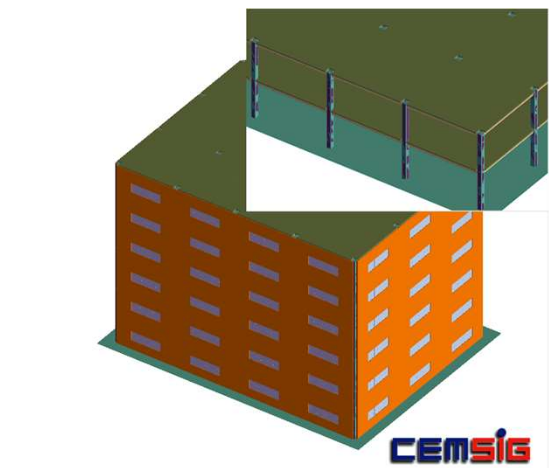
A structure shall be designed and executed in such a way that it will not be damaged by events such as:

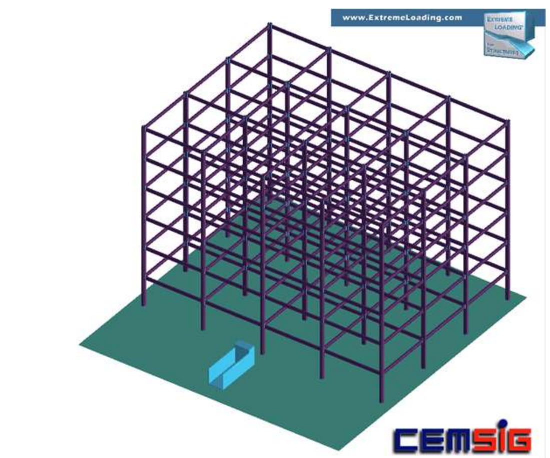
- explosion,

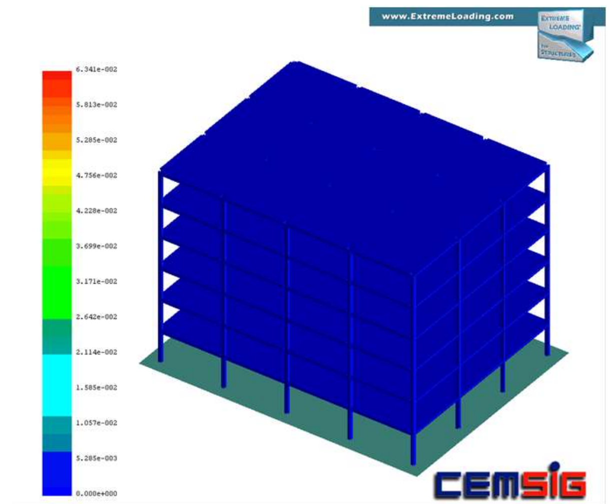
- impact, and

- the consequences of human errors,

to an extent disproportionate to the original cause (EN 1990)







Scope:

- Structural integrity under accidental actions should be preserved
- Accidental design situation: design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure

Objectives:

- Structural response should be determined by appropriate methods
- Application of simple methods for low level of protection buildings (low consequence classes) and more complex for HLOP structures :
  - Indirect methods: minimum requirements, e.g. tying method
  - Direct method:
    - Design against gas explosions
    - Design against impact
    - Design against blast
    - Alternate path method APM
- Studied building: 3D steel frame structure

Standards and codes:

- EN1090
- EN1991-1-7
- UFC 4-023-03 Design of buildings to resist progressive collapse, Department of Defense, USA
- Calculation of blast loads for application to structural components, European Commission, Joint Research Centre, Institute for the Protection and Security of the Citizen
- UFC 3-340-02, Structures to resist the effects of accidental explosions, Department of Defense, USA
- Donald O. Dusenberry, Handbook for blast-resistant design of buildings, 2010, John Wiley & Sons.
- Biggs, J.M. (1964), “Introduction to Structural Dynamics”, McGraw-Hill, New York.

Introduction and definitions

Definitions

- Robustness
- Progressive collapse
- Structural integrity
- Accidental loading, Exceptional loading
- Class of consequences ➔ method of analysis
  - Indirect method: minimum requirements, e.g. tying method
  - Direct method:
    - Design against gas explosions
    - Design against impact
    - Design against blast
    - Alternate path method APM

Robustness & Progressive collapse

Robustness - the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent *disproportionate* to the original cause.

EN 1991-1-7

Progressive collapse - the spread of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as *disproportionate collapse*.

ASCE 7-05

Robustness

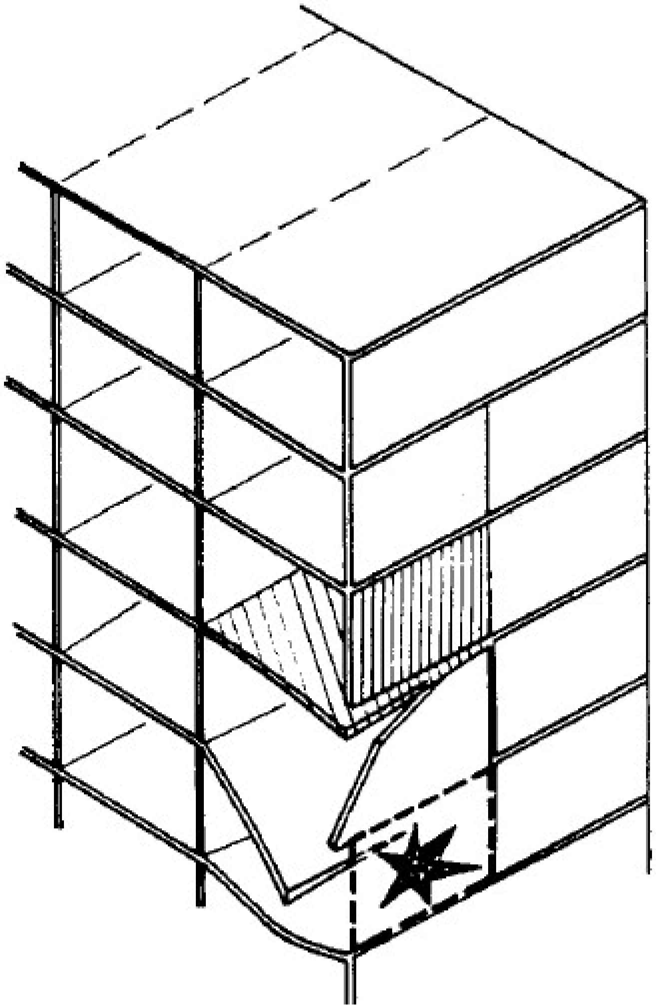
Robustness is required to resist to extreme events such as:
- explosion
- terrorist attack
- impact
- fire*
- earthquake*

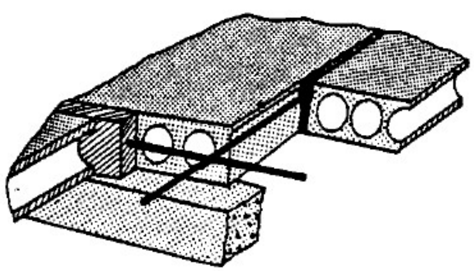
Such events have low probability but sometimes disastrous consequences

The Cardington Fire Test

* fire or earthquake events not foreseen when initially designing the structure

U.K. recommendations





RONAN POINT, 1968

- Collapse at Ronan Point, Canning Town, England, 16th May 1968:
    - caused by a gas explosion in the corner of the 18th floor
    - progressive collapse (precast concrete slab elements)

First provisions: HMSO (1976).
Statutory Instrument, No. 1676:
Building and Buildings, London

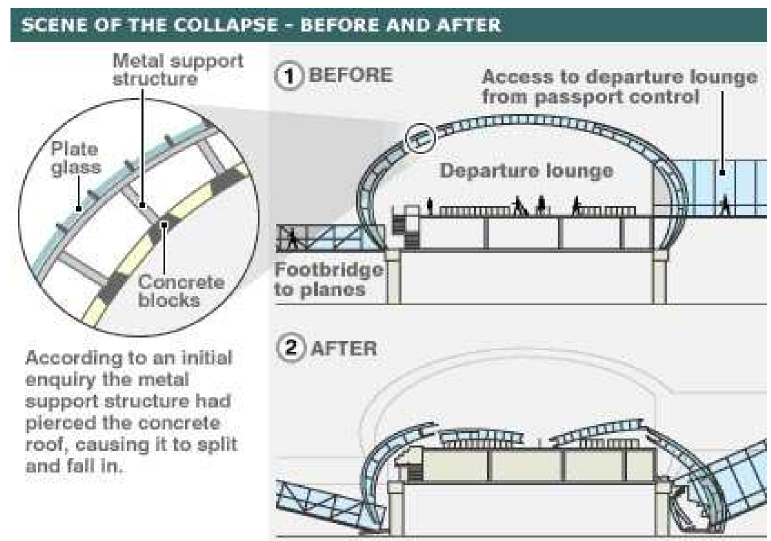
Progressive collapse

OKLAHOMA CITY, 1995

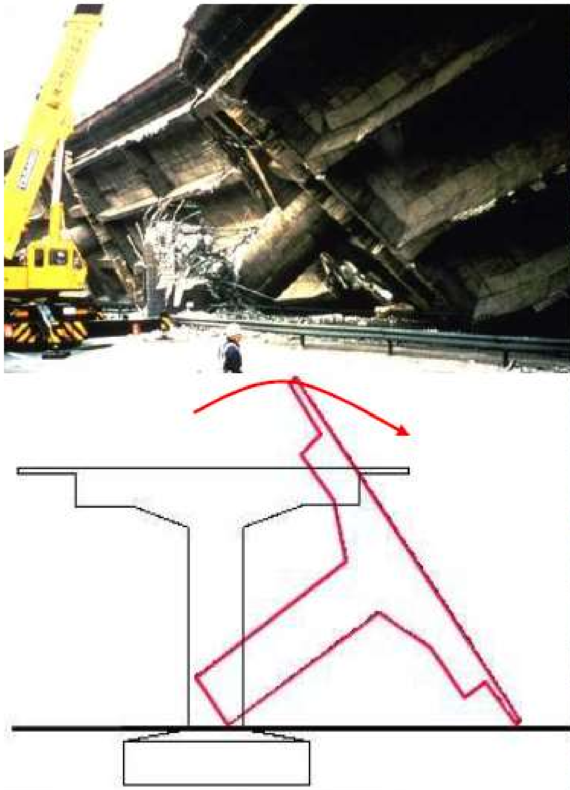
WTC, 2001

Kattowitz, Poland

Wrong steel-concrete connection details at Charles-de-Gaulle Airport new terminal, 2004



COLLAPSE OF HANSHIN EXPRESSWAY – KOBE, 1995
initial collapse

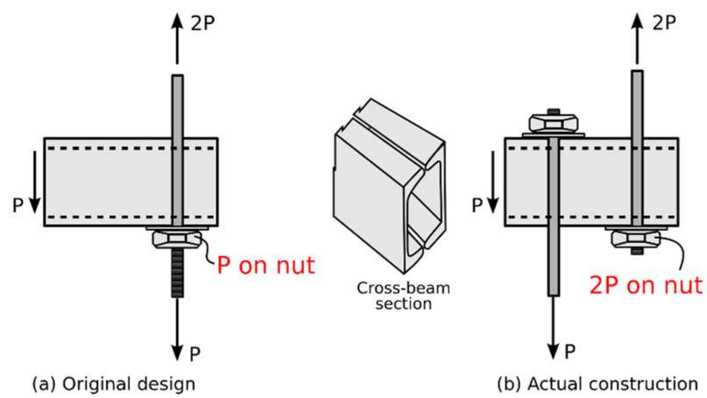


Structural integrity

- In the literature, the term “structural robustness” is occasionally replaced by “structural integrity” but, actually, the latter has a different meaning
- It indicates the wholeness and intactness of a structure after an extreme event.

- Structural integrity is the ability of a structural component or a structure to resist the loads, without breaking or deforming excessively.
- It assures that the construction will perform its designed function during reasonable use, for as long as its intended life span.
- Items are constructed with structural integrity to prevent catastrophic failure, which can result in injuries, severe damage, death, and/or monetary losses.





Reinforced concrete column without/with composite wrap

Loading situations

- Snow,
- Wind (storm, …),
- Earthquakes
- Tsunami,
- …

Natural hazard

- Impacts,
- Explosions, …
- …

Human-made hazard

Loading situations

- Permanent (e.g. dead load)
- Snow,
- Wind (wind storms, …)
- Earthquakes

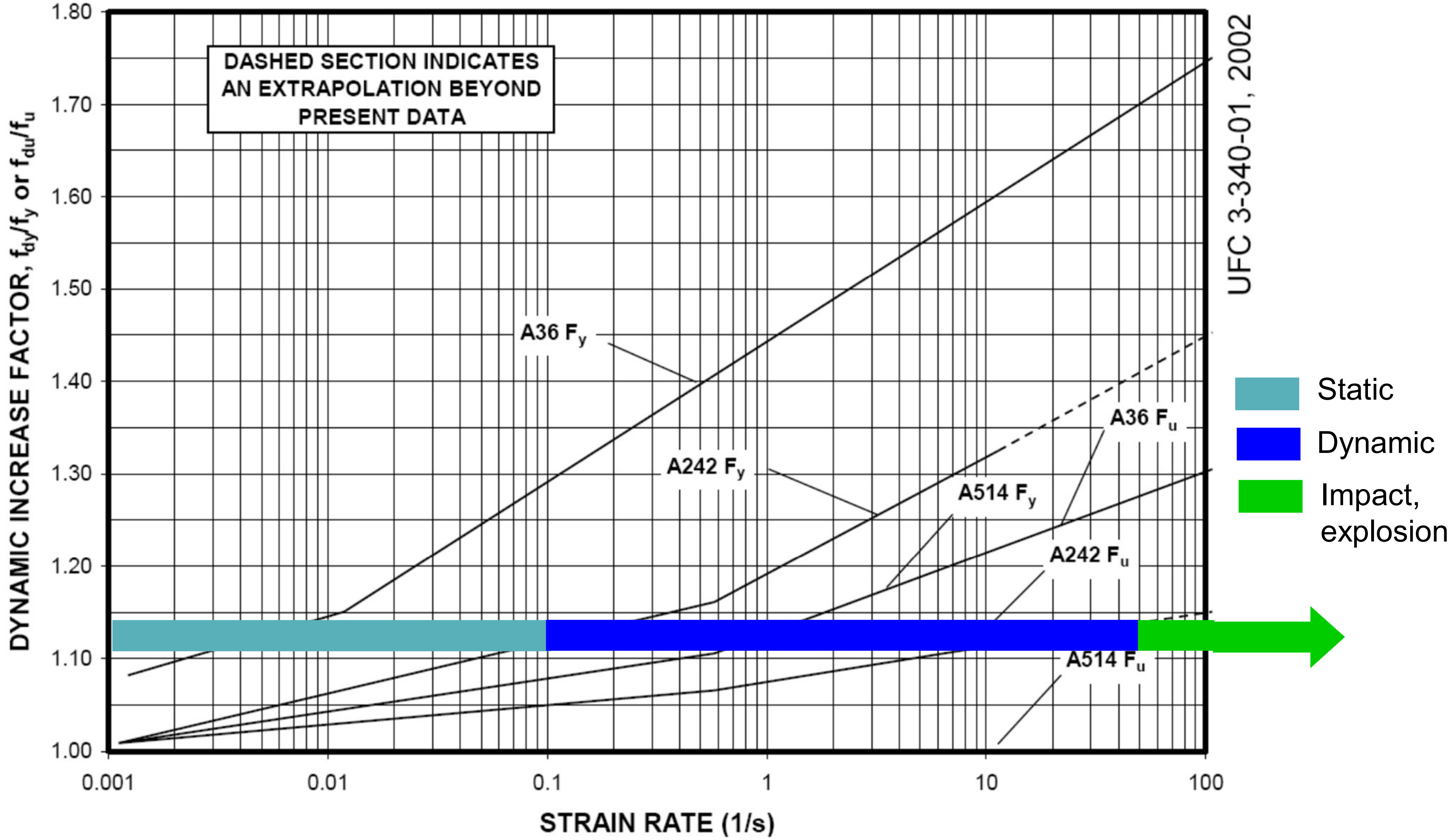
- Fire
- Impact
- Explosion
- Blast
-

Different types:

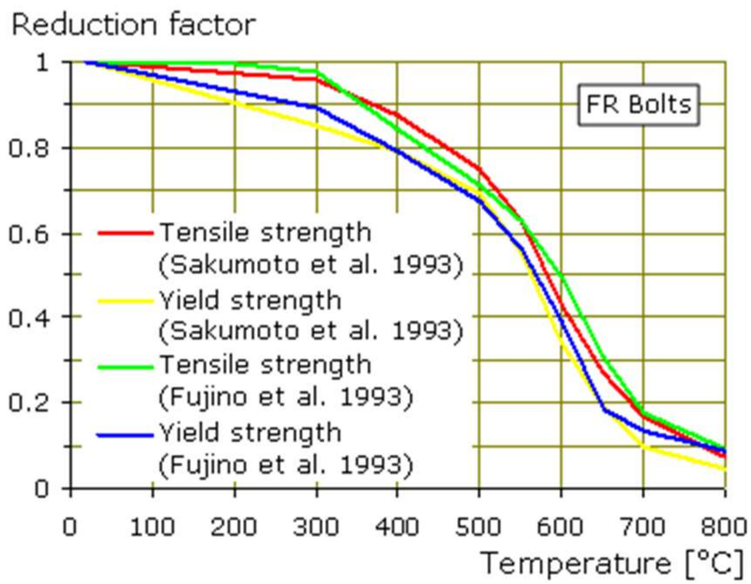
- **quasi-static**
- **dynamic / impact**

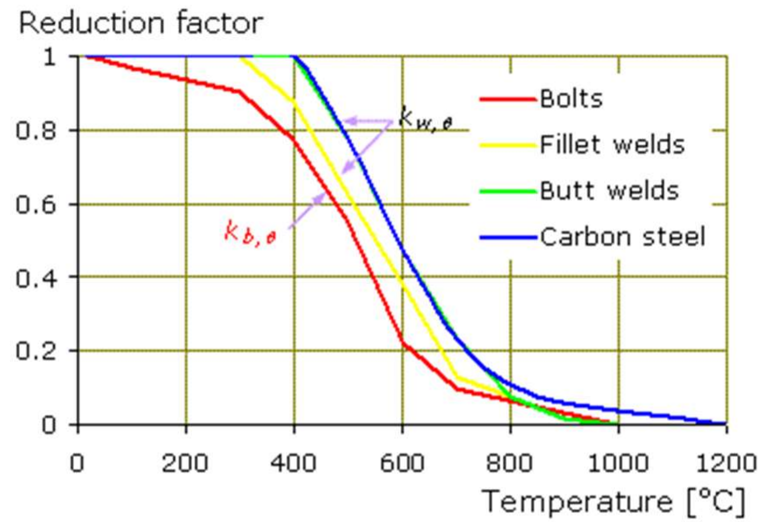
- **monotonic**
- **cyclic**

- Induced deformations, material property change

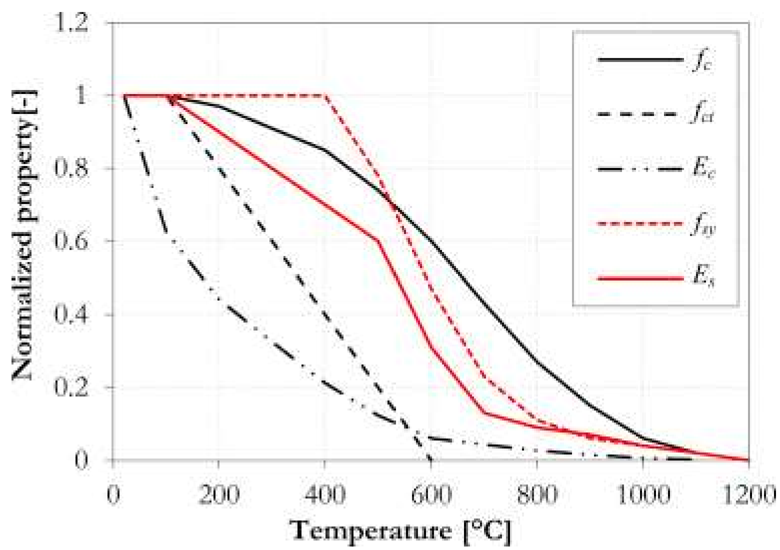


Strain Rate Factors for Structural Steel

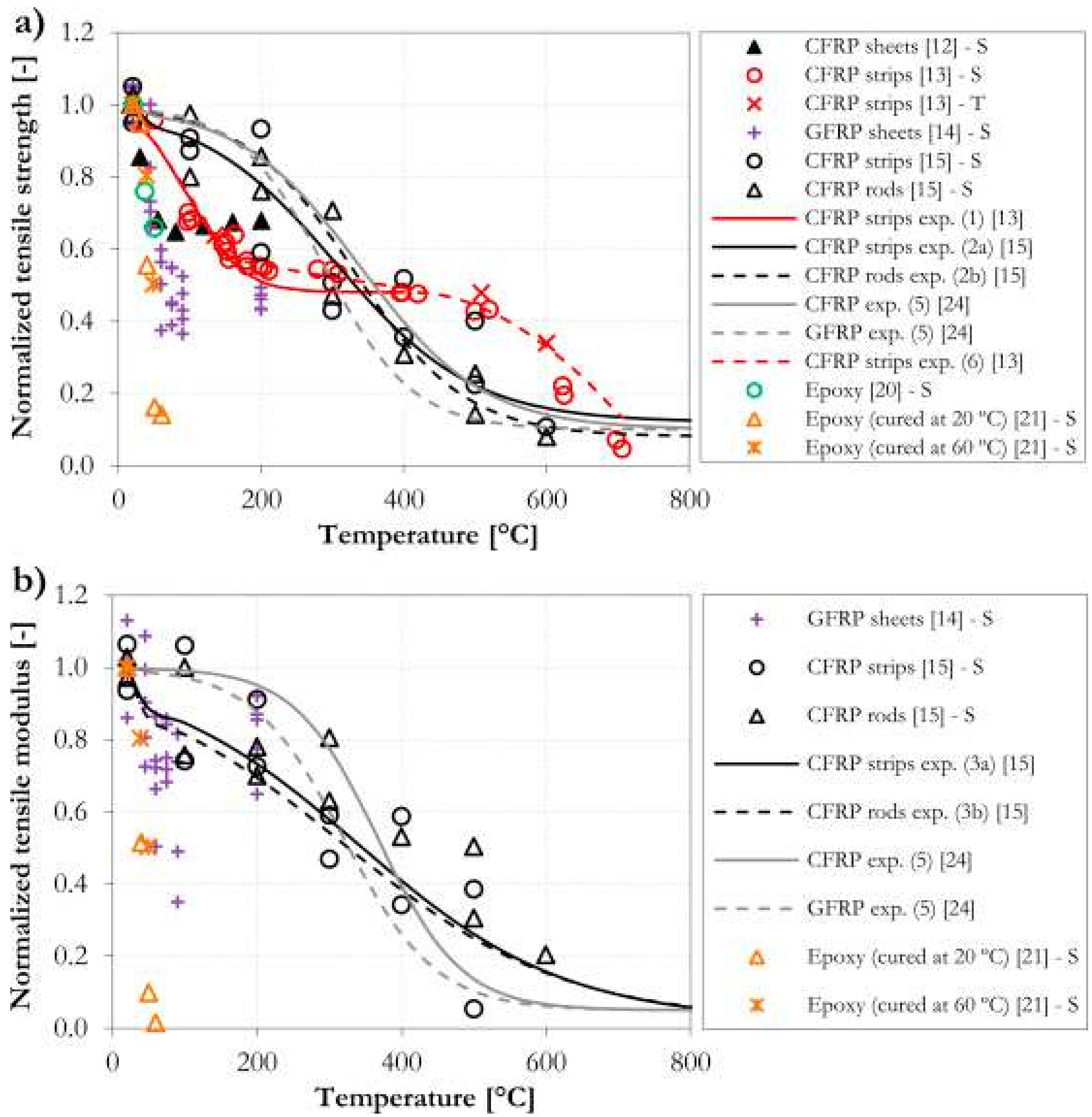




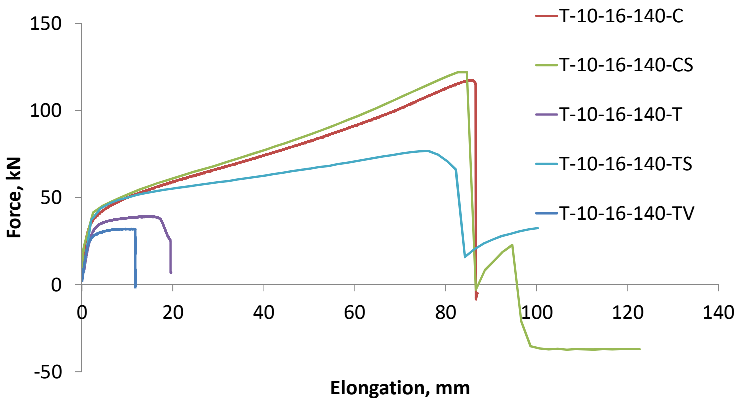
Strength reduction factor



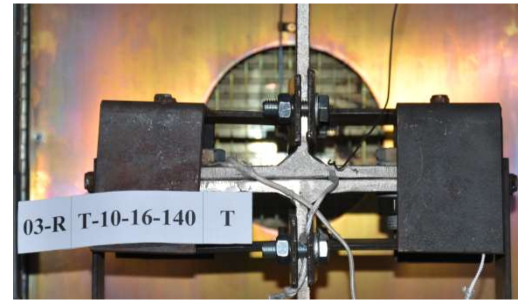
Typical mechanical properties of concrete (compressive strength, fc, tensile strength, fct, and elastic modulus, Ec) and steel (yielding stress, fsy, and elastic modulus, Es) as a function of temperature, according to Eurocode 2 Part 1–2



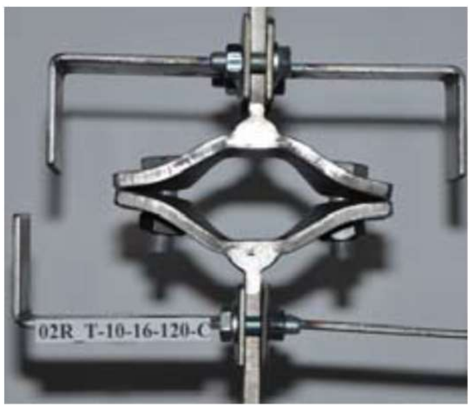
Normalized tensile properties vs. temperature of FRPs and epoxy adhesives commonly used to strengthen RC elements: a) tensile strength; b) elastic modulus (S-steady-state conditions; T-transient conditions).



Initial



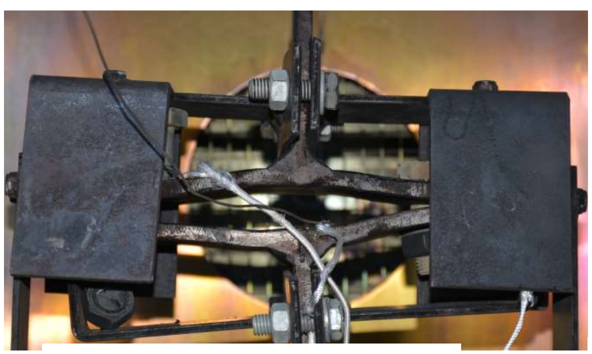
Initial, T=542°C



cvasistatic

v_1 = 10 mm/sec

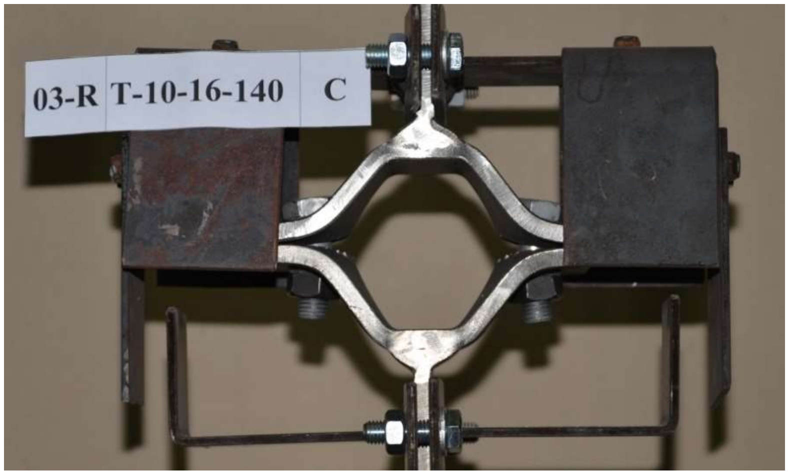
v2 = 15 mm/sec

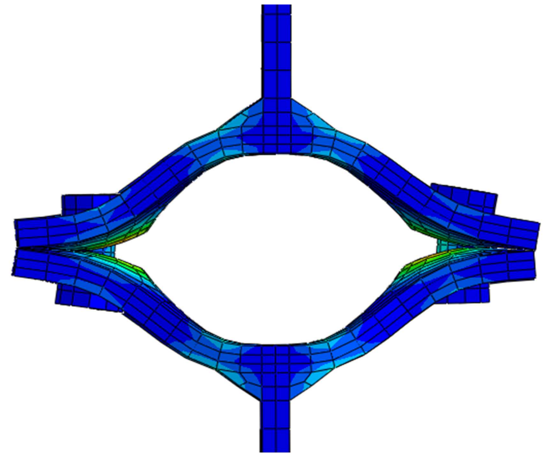


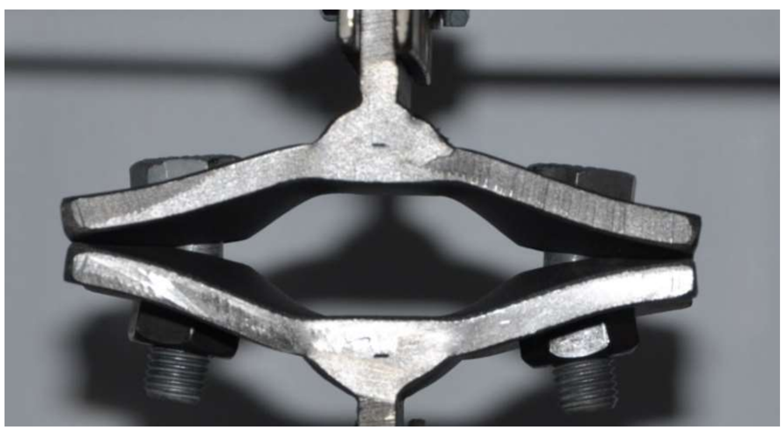
cvasistatic, T=542°C

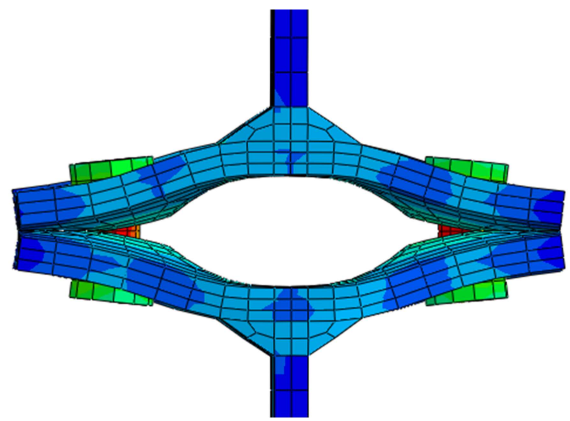
v1 = 10 mm/sec, T=542°C

v2 = 15 mm/sec, T=542°C

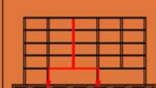
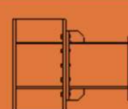

03-R T-10-16-140 C







Multi-hazard design matrix
(adapted from FEMA 577)

	Site and building characteristics		Hazard: Seismic	Flood	Wind	Fire	Explosion	Interaction
1	Elevated building site		-	+	0	0	+	Highly beneficial for floods and external bomb explosion, not significant for wind or fire
2	Re-entrant corner plan forms		-	0	-	0	-	Stress concentration at corners, irregular behavior in case of earthquakes; localized wind pressures, amplification of shock wave in case of external blast
3	Very irregular buildings		-	0	-	-	-	Indirect load paths, stress concentrations in earthquakes, explosions. Localized high wind pressure, aggravates evacuation in case of fire
4	Large roof overhangs		-	0	-	-	-	Vulnerable to earthquakes (vertical motion), wind and also adjacent external blast. Mai pose risk also in case of fire evacuation
5	Steel structural frame		+	+	+	-	+	When properly detailed, is recommended in seismic and high-wind zones. Good in flood with proper detailing. Vulnerable to fire if is not protected or well detailed and designed. Low vulnerability in case of blast and explosion, offers multiple paths.
6	Indirect load path		-	0	-	-	-	Very vulnerable for seismic, wind and explosion hazards because poor structural integrity increases likelihood of collapse. Fire may further weaken structure.
7	Ductile detailing of structure and connections		+	0	+	+	+	Provides good plastic response. The structure has large ductility and is more resistant to collapse in case of extreme loading

Note: The following convention has been used in the table:
+ indicates a desirable condition or beneficial interaction between the designated component/system and hazard
0 indicates little or no significant interaction between the designated component/system and hazard
- indicates an undesirable condition or the increased vulnerability of a designated component/system to a hazard

Eurocode EN 1990 « Basis of structural design »

(P) A structure shall be designed and executed in such a way that it will not be damaged by events such as :

- – explosion,
- – impact, and
- – the consequences of human errors, to an extent disproportionate to the original cause.

NOTE 1 The events to be taken into account are those agreed for an individual project with the client and the relevant authority.

NOTE 2 Further information is given in EN 1991-1-7.

Eurocode and U.K. recommendations

Rules in EN 1991-1-7 Accidental Actions and UK Codes of Practice:

- introduce horizontal and vertical ties

- design key elements for a recommended accidental design action (ex. in case of gas explosion A_d= 34 kN/m2)

- ensure that upon the notional removal of a supporting column, wall section or beam, the damage does not exceed 15% of the floor in each of 2 adjacent storeys (alternate path method)

<u>Consequence Class</u>

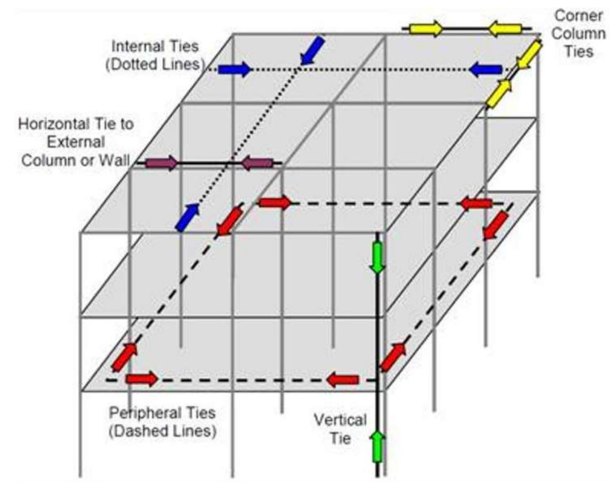
Consequences Class	Description	Examples of buildings and civil engineering works	
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)	Significant demand in terms of robustness
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)	
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses	Low or no demand in terms of robustness

Depending on CC, various philosophies to follow:

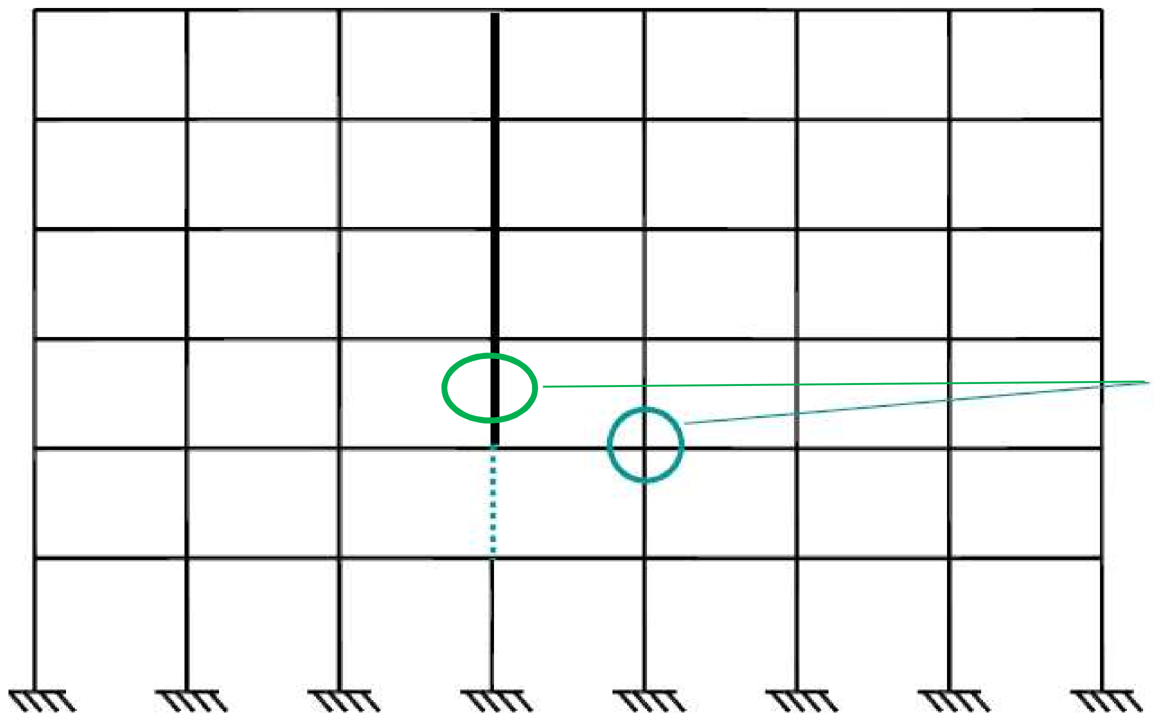
- Indirect methods:
  - No scenario considered
  - Just design requirements to fulfill

    Ex: **Tying resistance method**
- Direct methods:
  - **Specific load resistance method**
    - **Impact**
    - **Explosion**
    - **Blast**
  - **Alternative load path method**

Indirect methods



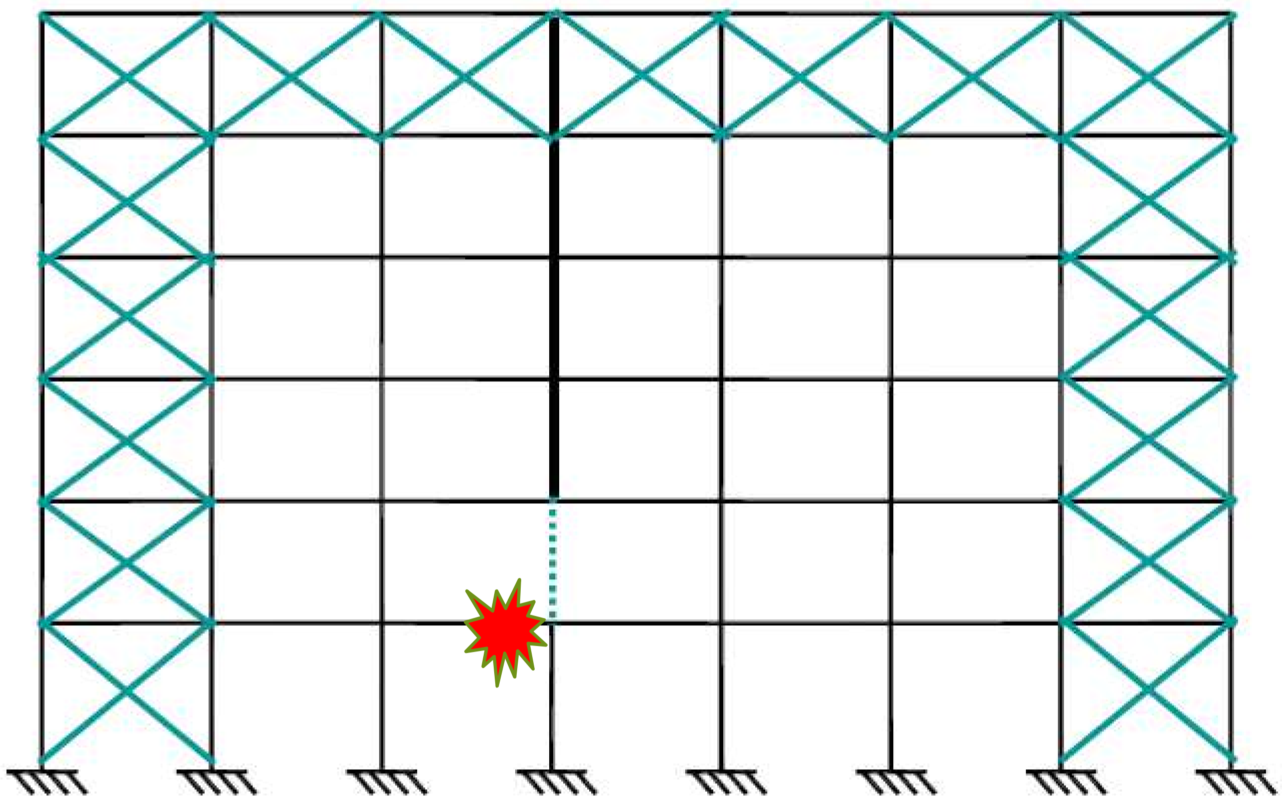
DoD, 2016

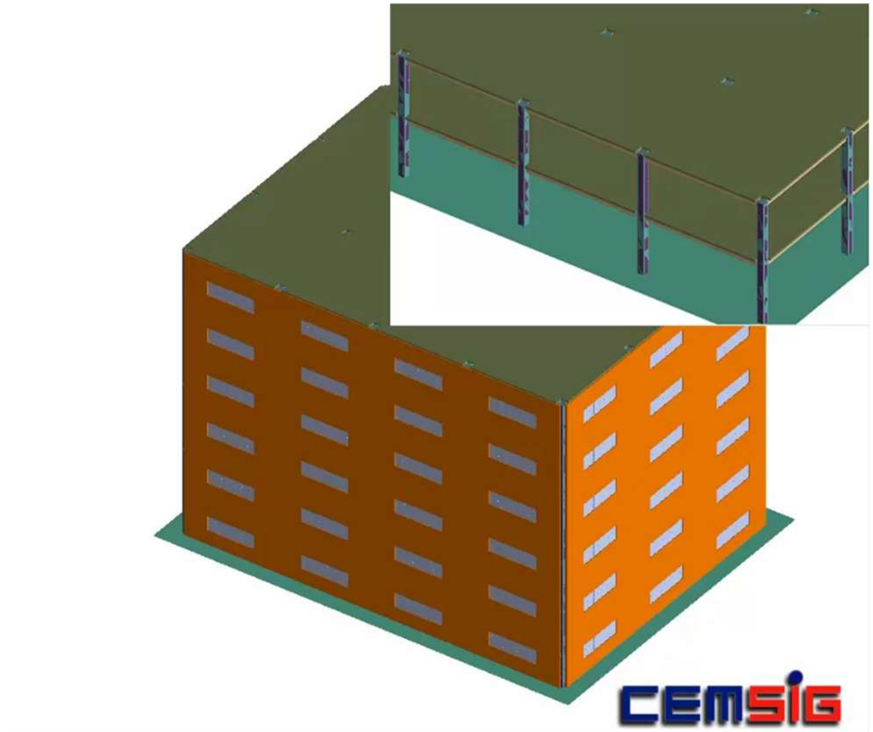


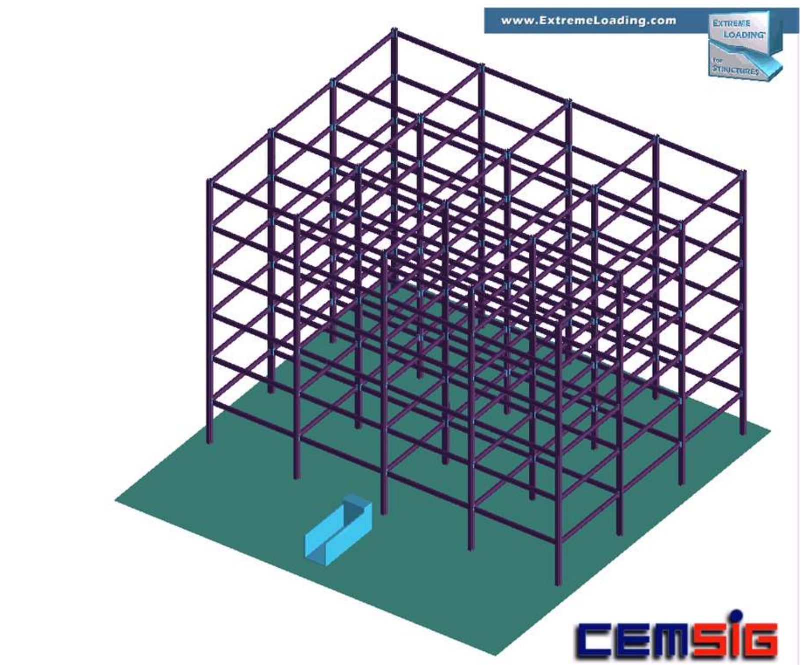
« TYING RESISTANCE » REQUIREMENT

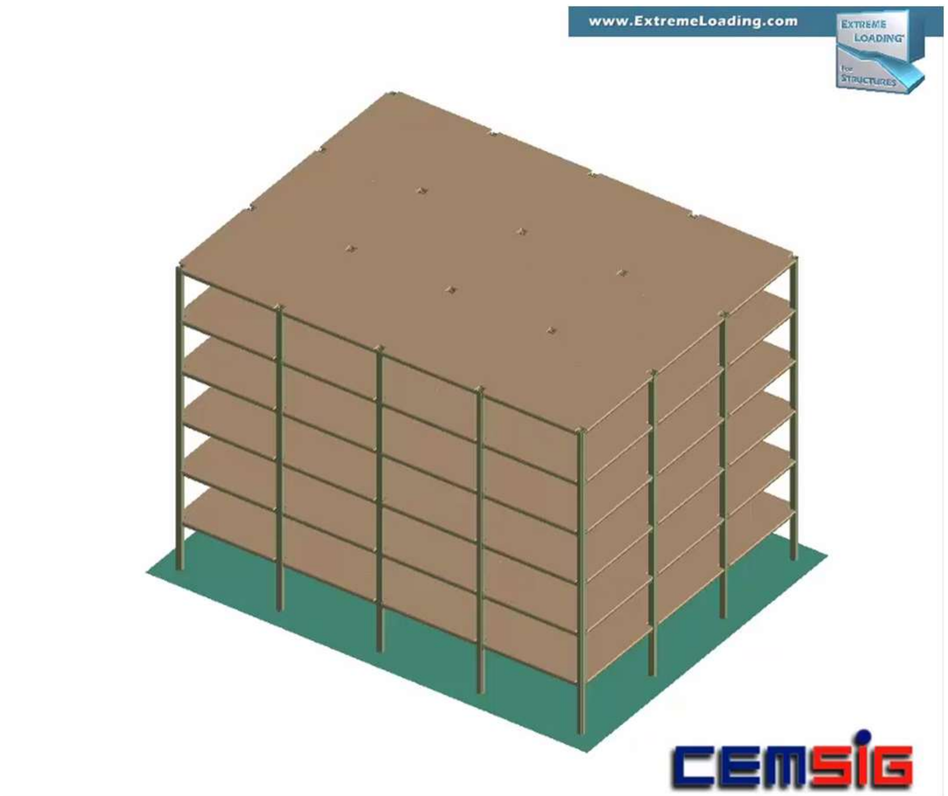
Direct methods

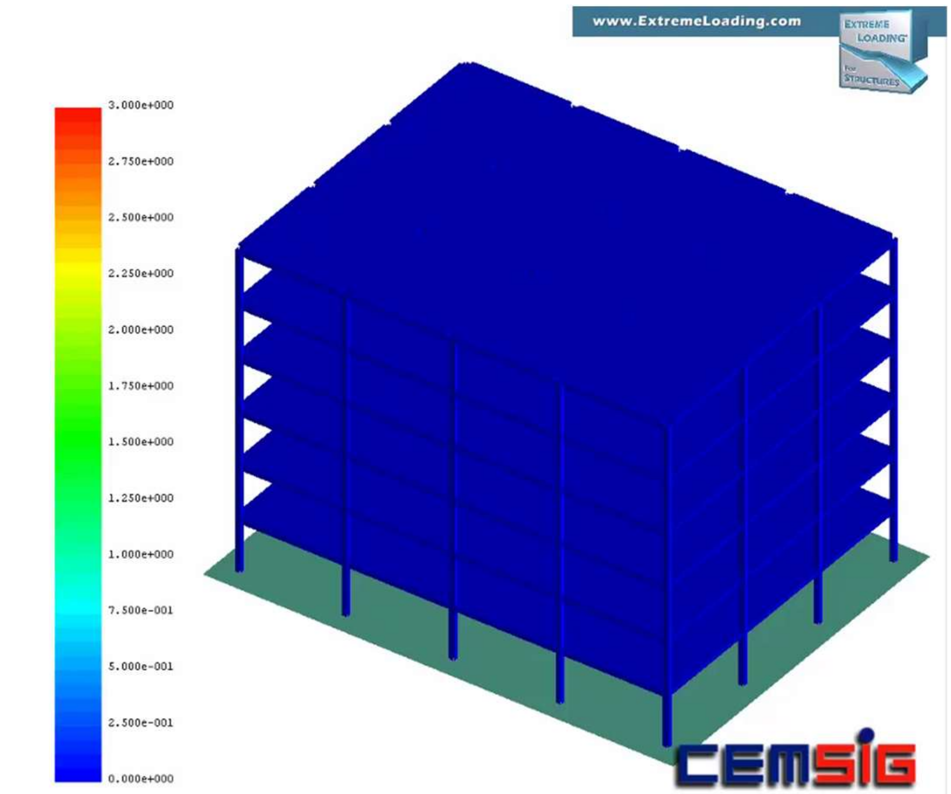
Specific load resistance




CEMSIG


www.ExtremeLoading.com
CEMSIG


www.ExtremeLoading.com
EXTREME LOADING FOR STRUCTURES
CEMSIG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EXTREME LOADING FOR STRUCTURES
3.000e+000
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1.500e+000
1.250e+000
1.000e+000
7.500e-001
5.000e-001
2.500e-001
0.000e+000
CEMSIG

Tying resistance (EN1991-1-7):

1. Effective horizontal ties
2. Effective vertical ties

1. Effective horizontal ties (framed buildings)

- Effective horizontal ties should be provided around the perimeter of each floor and roof level and internally in two right angle directions to tie the column and wall elements securely to the structure of the building.
- At least 30 % of the ties should be located within the close vicinity of the lines of columns and walls.
- Each continuous tie, including its end connections, should be capable of sustaining a design tensile load of “Ti” for the accidental limit state in the case of internal ties, and “Tp” , in the case of perimeter ties, equal to the following values:

for internal ties, $T_i = 0{,}8\,(g_k + \psi q_k)sL$ or 75 kN, whichever the greater.

for perimeter ties $T_p = 0{,}4\,(g_k + \psi q_k)sL$ or 75 kN, whichever the greater

Where :

s is the spacing of ties.

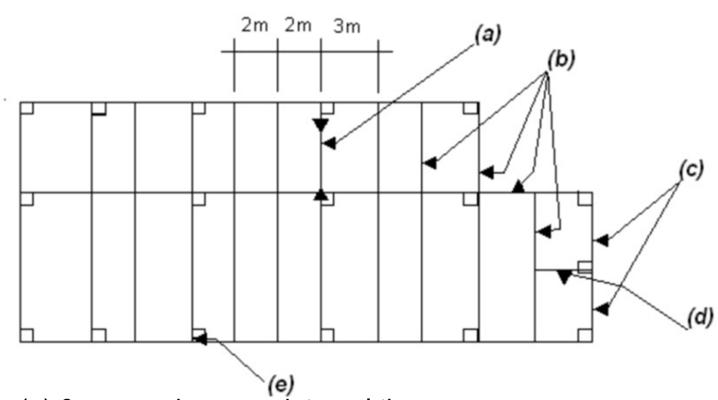
L is the span of the tie.

ψ is the factor according to the accidental load combination (ie. ψ_1 or ψ_2).

EXAMPLE Calculating the accidental design tensile force T_i in 6 m span beam.

Characteristic loading : q_k = 5, 0 kN/m^2 and g_k = 3, 0 kN/m^2

$$T_i = 0{,}8\,(3{,}00 + 0{,}5 \times 5{,}00)\frac{3+2}{2} \times 6{,}0 = 66\,kN$$ (being less than 75 kN)



(a) 6 m span beam as internal tie
(b) All beams designed to act as ties
(c) Perimeter ties
(d) Tie anchoring column
(e) Edge column

2. Effective vertical ties

- Each column and wall should be tied continuously from the foundations to roof level.
- In the case of framed buildings (*e.g.* steel structures) the columns and walls carrying vertical actions should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with normal loading.

Natural gas explosions (EN 1991-1-7)

For buildings provided for having natural gas installed, the structure may be designed to withstand the effects of an internal natural gas explosion using a nominal equivalent static pressure given by expressions:

$$P_d = 3 + P_{stat}$$

or

$$\mathrm{P_d} = 3 + \mathrm{P_{stat}}/2 + 0{,}04/(\mathrm{A_v}/\mathrm{V})^2$$

whichever is the greater

where:
P_{stat} is the uniformly distributed static pressure at which venting components will fail, in (kN/m^2);
Av is the area of venting components, in m^2
V is the volume of rectangular enclosure [m^3].

Where building components with different p_{stat} values contribute to the venting area, the largest value of p_{stat} is to be used. No value p_{d} greater then 50 kN/m^2 need to be taken into account.

For buildings where provision of natural gas is totally impossible, a reduced value of the equivalent static pressure $\mathrm{p_d}$ may be appropriate.

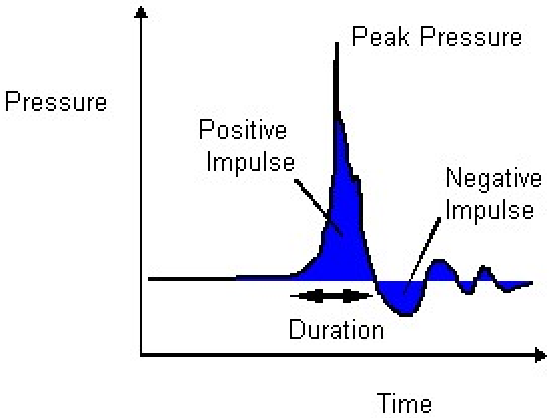
Key Elements (local resistance method)

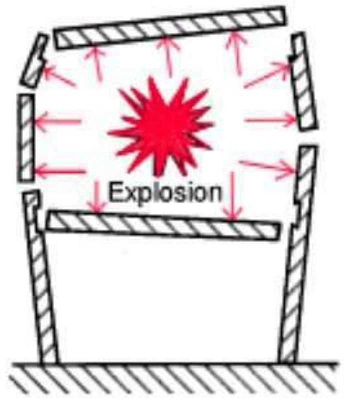
key element: a structural member upon which the stability of the remainder of the structure depends.

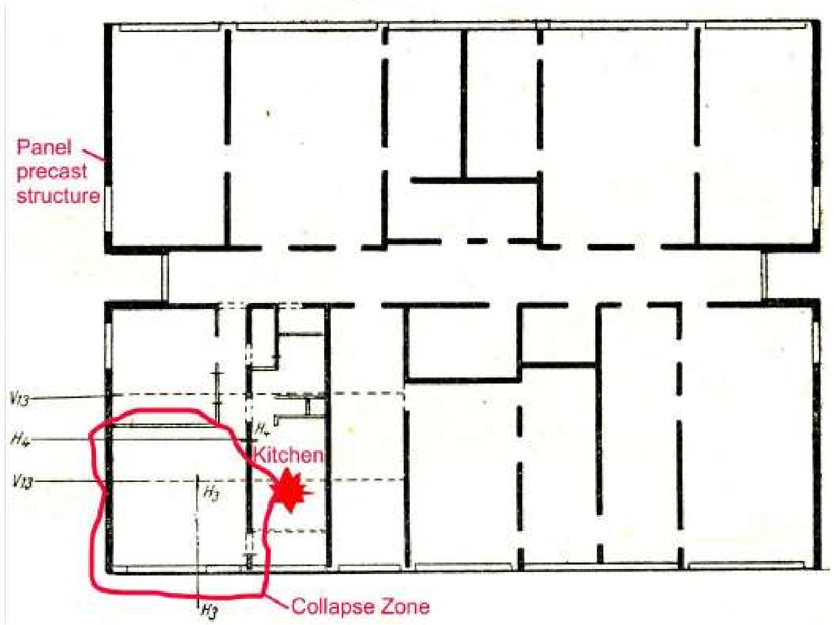
- For building structures a "key element" should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the ultimate strength of such components and their connections.
- Such accidental design loading should be assumed to act simultaneously with normal loading

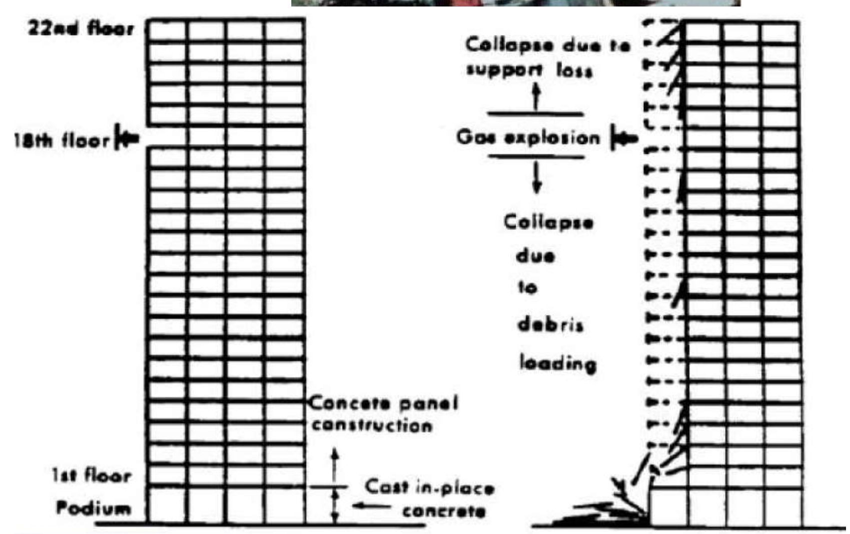
NOTE
The National Annex may give a value for A_d. The recommended value of A_d for building structures is 34 kN/m^2

EXPLOSIONS
Inner explosion – ROMAN POINTBUILDING, England 1968









14th of September 2007

- Gas accumulated in the basement, then went up on the staircase and first floor apartments
- First floor slab collapsed and fall.
- Doors from upper stories dislocated.
- Building remained unstable and was demolished

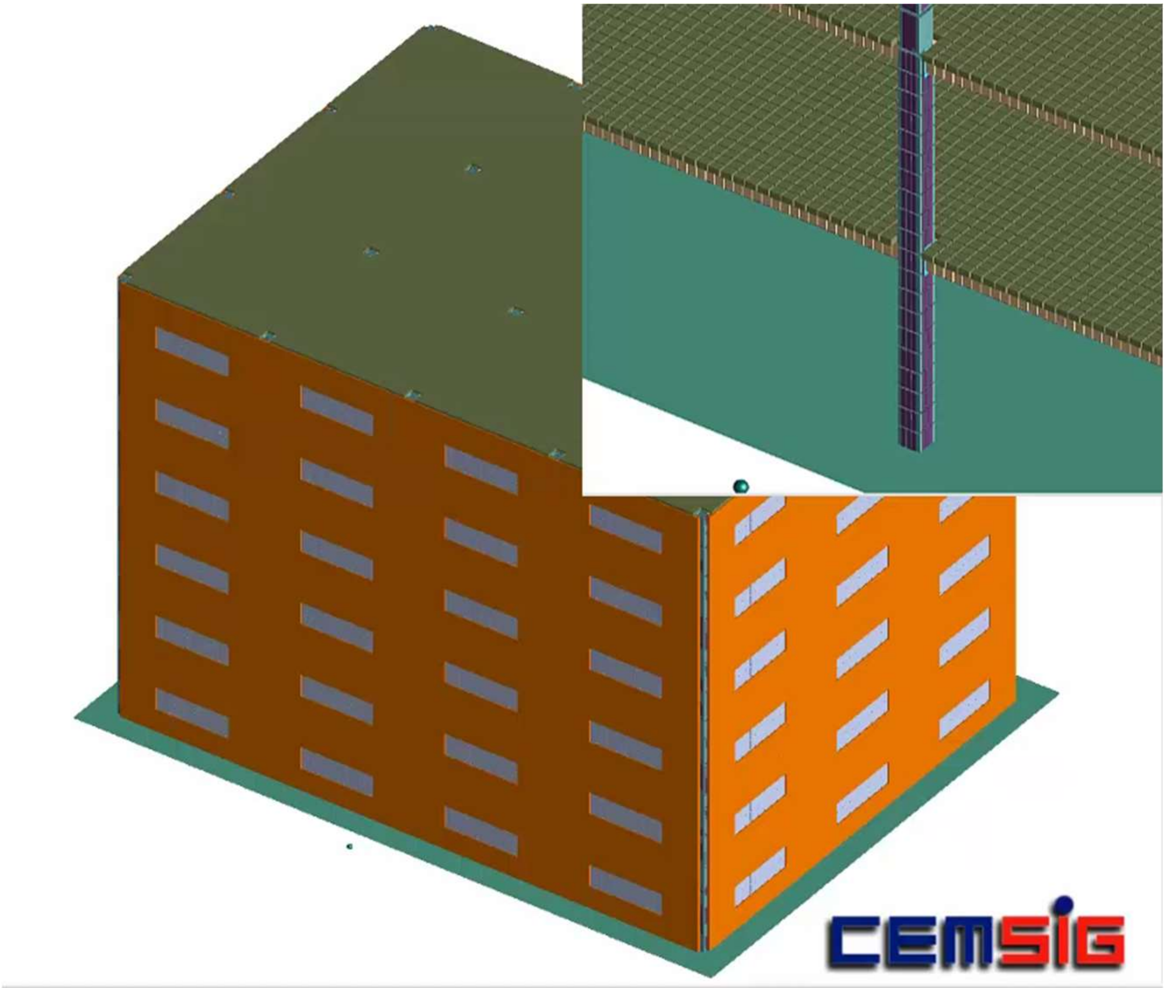
1st of October 2013

- Gas accumulated in an apartment at the 2nd floor
- The explosion completely destroyed the slab above 2nd storey and collapse falling on the floor below
- The building was heavily damaged

15th of September 2017

March 4, 2018, Apartment building collapse in Poznan, western Poland, due to gas explosion

No provisions in EN 1991-1-7 regarding external explosions!



Impacts

- Car or truck?
    - Location?
    - Speed and mass?
- Plane?
    - Location?
    - Speed and mass?
- …

Dynamic design for impact (general case)

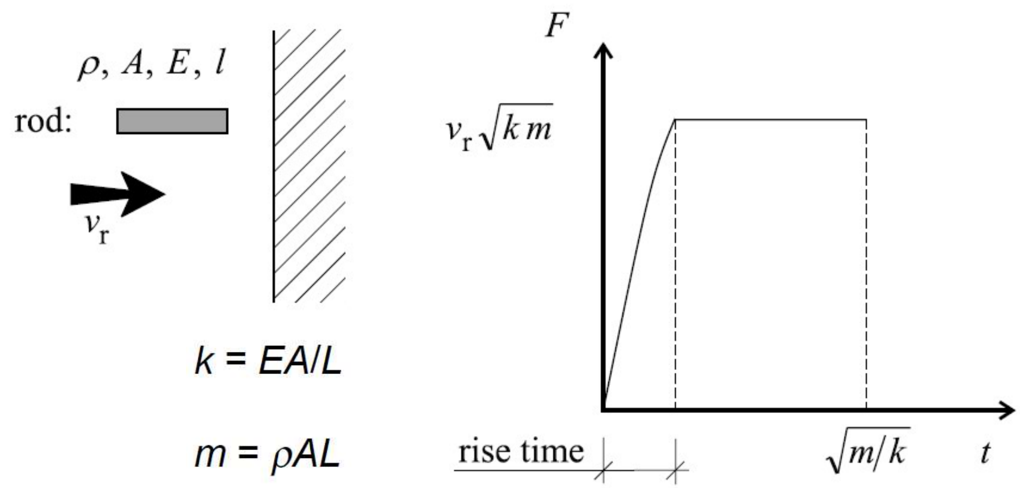
Impact is characterized as either *hard impact*, when the energy is mainly dissipated by the impacting body, or *soft impact*, when the structure is designed to deform in order to absorb the impact energy.

Hard impact

If the structure is rigid and immovable and the colliding object deforms linearly, during the impact phase and remains rigid during unloading, the maximum resulting dynamic interaction force is given by expression:

$$F = v_r \sqrt{k\, m}$$

- v_r is the object velocity at impact;
- k is the equivalent elastic stiffness of the object (i.e. the ratio between force F and total deformation);
- m is the mass of the colliding object



Impact model, *F* = dynamic interaction force

Soft impact

- If the structure is assumed elastic and the colliding object rigid, the expressions given for rigid impact still apply and should be used with *k* being the stiffness of the structure.
- If the structure is designed to absorb the impact energy by plastic deformations, it should be ensured that its ductility is sufficient to absorb the total kinetic energy $\frac{1}{2} m v_r^{2}$ of the colliding object.
- In the limit case of rigid-plastic response of the structure, the above requirement is satisfied by the condition of expression:

$$\frac{1}{2} m v_r^{2} \leq F_o y_o$$

where
F_o is the plastic strength of the structure, i.e. the quasi-static limit value of the force F;
y_o is its deformation capacity, i.e. the displacement of the point of impact that the structure can undergo.

Impact of a vehicle

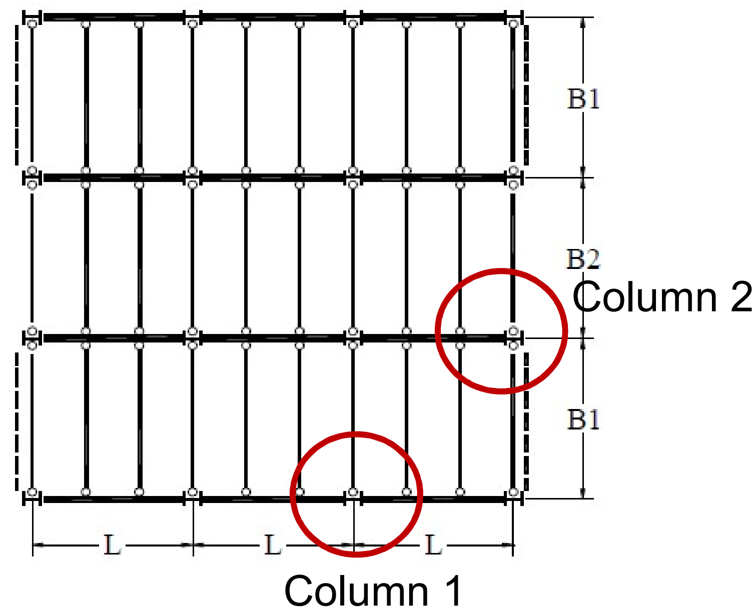
- Structures for which the energy is mainly dissipated by the impacting object can be calculated using the following data:

Caterory	Minimum Force $F_{d,x}$ [a] [kN]	Minimum Force $F_{d,y}$ [a] [kN]
Motorways and country national roads	1000	500
Country Roads in rural area	750	375
Roads in Urban area	500	250
Court yards and parking garages with access to: - Cars - Lorries [b]	 50 150	 25 75

[a] x = direction of normal travel, y = perpendicular to the direction of normal travel.

[b] The term 'lorry' refers to vehicles with maximum gross weight greater than 3.5 ton.

- Recommended equivalent static loads for the design
- Fd,x and Fd,y have not to be considered at same time
- *h*, position of the impact load Fd , varying from *0,5 m* (cars) to *1,5 m* (trucks)

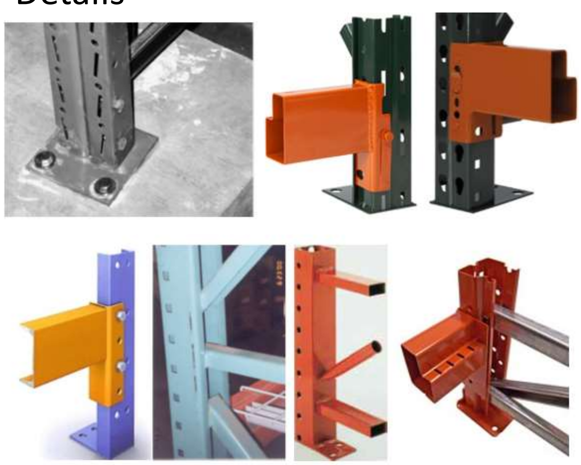


Case study: Impact of a forklift truck with a selective palleted racks

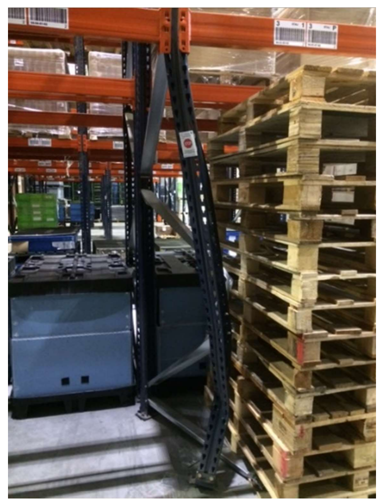
- Structures for which the energy is mainly dissipated by the impacting object can be calculated using the following data:

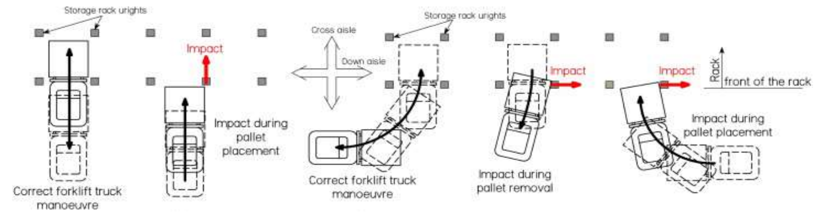
Selective pallet rack systems

Details

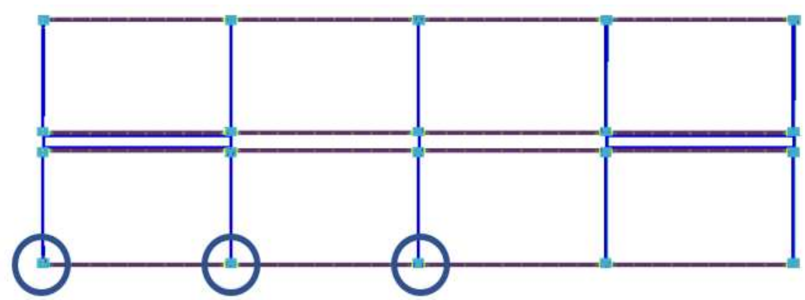


Accidental collision of forklift trucks

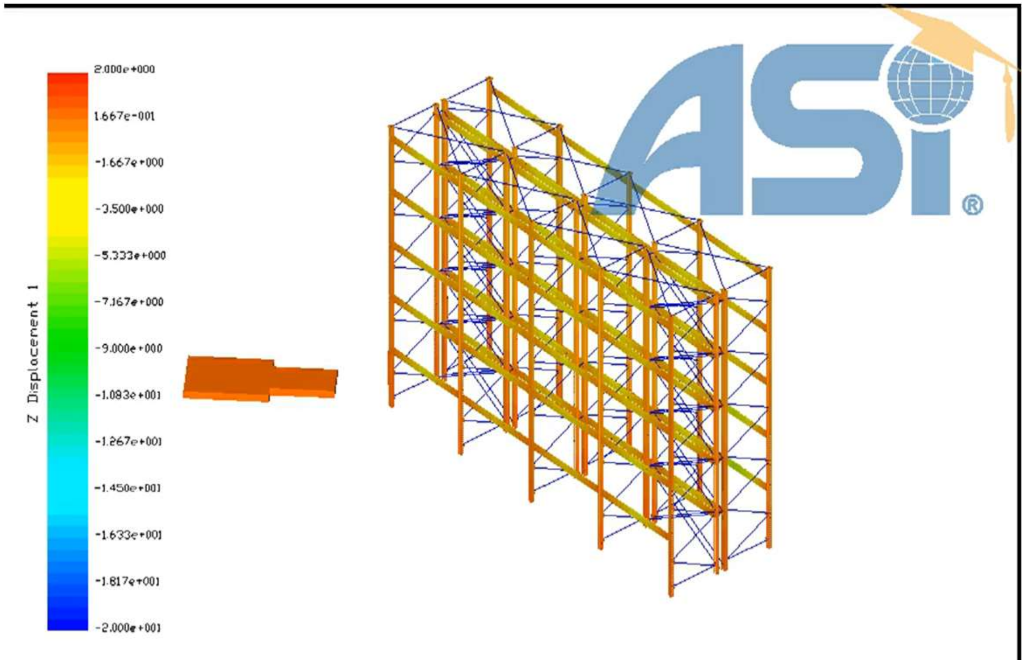


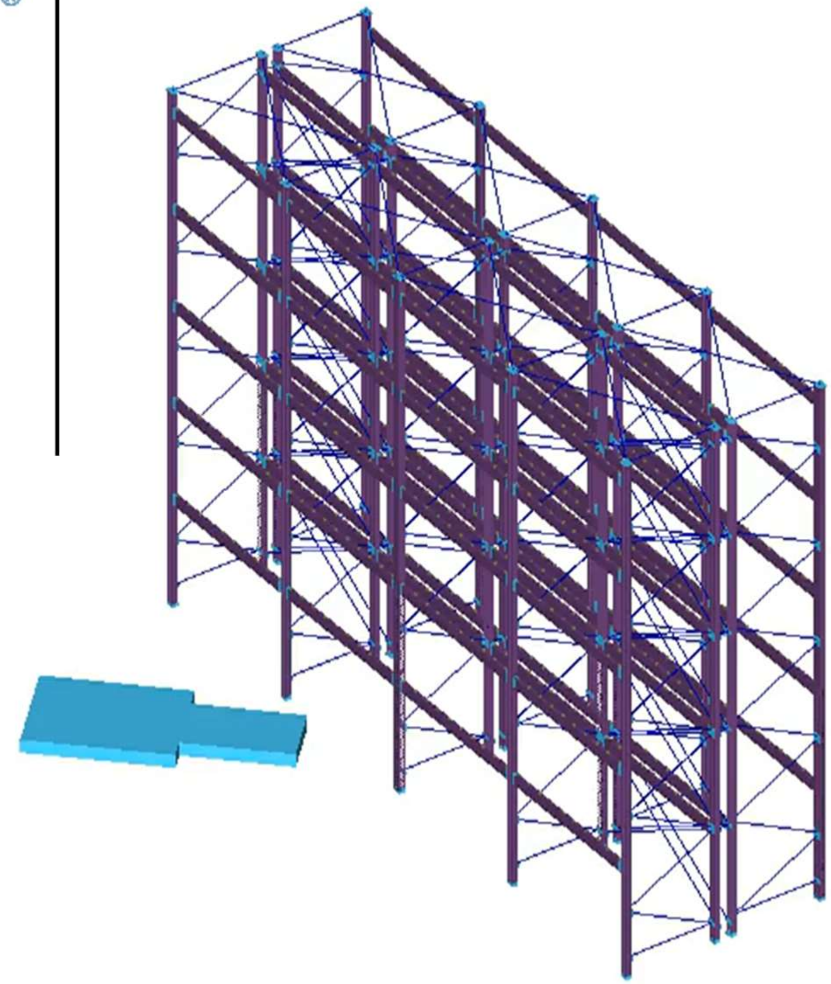


Position of uprights for loss/impact scenarios



Collapse propagation





Eurocode EN 1990 « Basis of structural design »

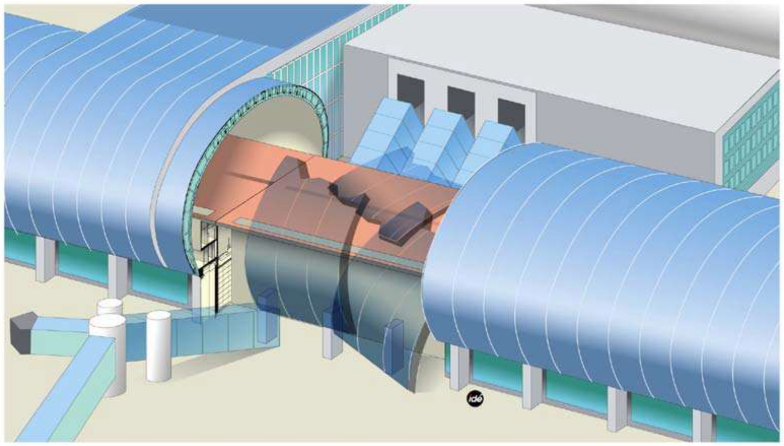
(P) A structure shall be designed and executed in such a way that it will not be damaged by events such as :

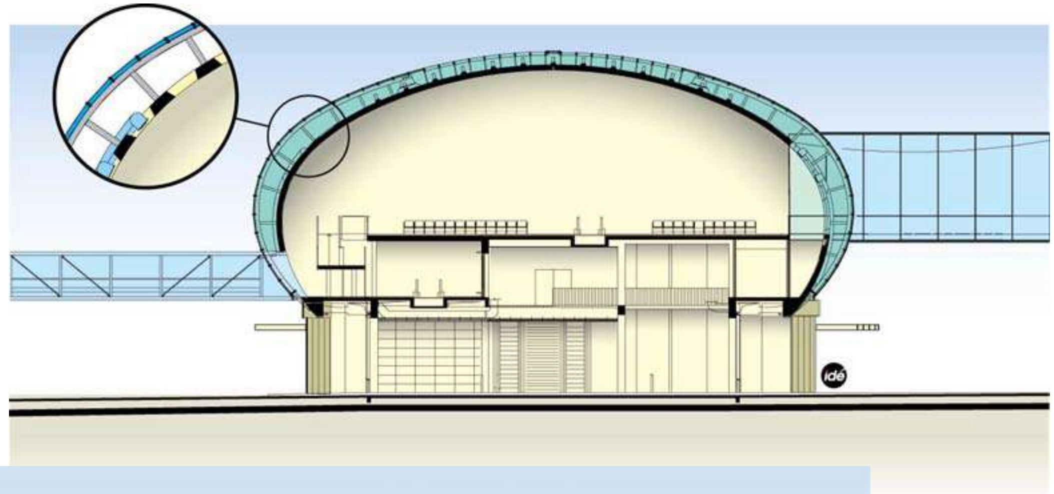
– explosion,
– impact, and
– the consequences of human errors, to an extent disproportionate to the original cause.

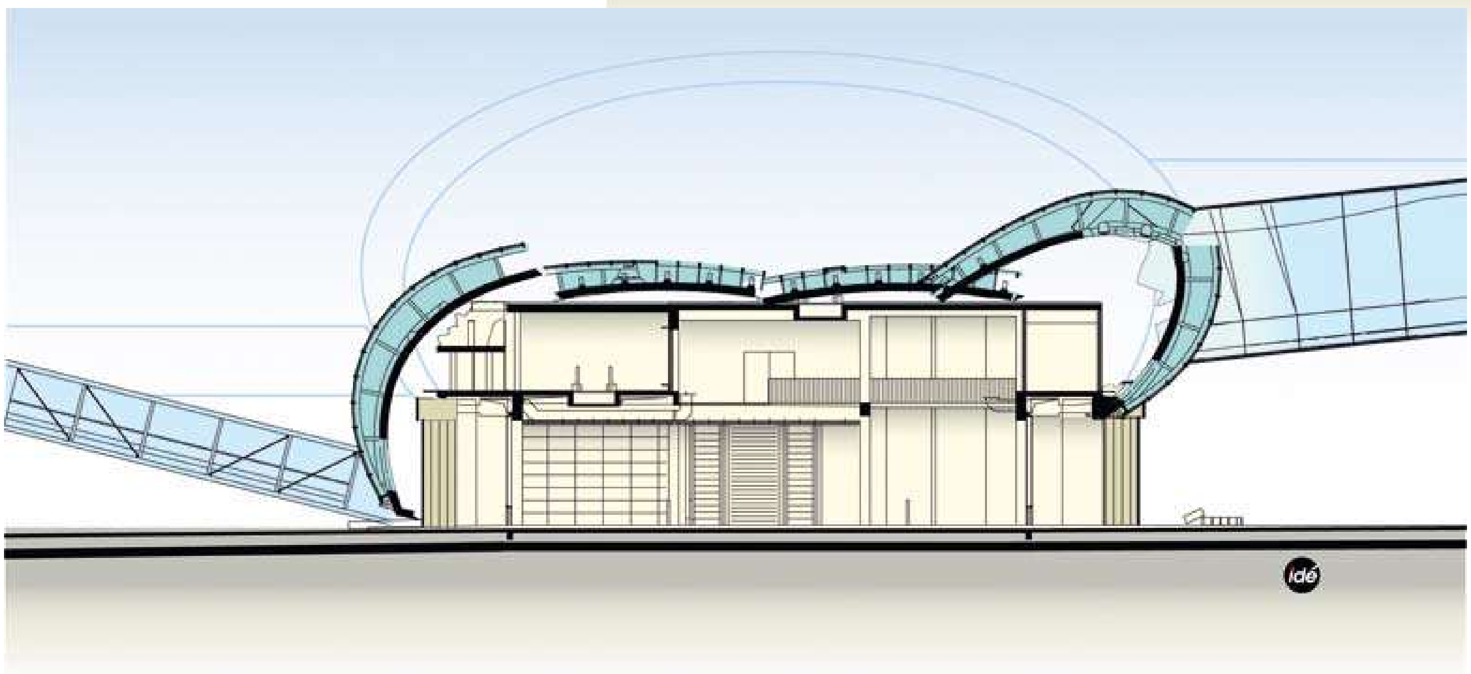
NOTE 1 The events to be taken into account are those agreed for an individual project with the client and the relevant authority.

NOTE 2 Further information is given in EN 1991-1-7.

Collapse of Paris Airport Terminal 2E, 2004



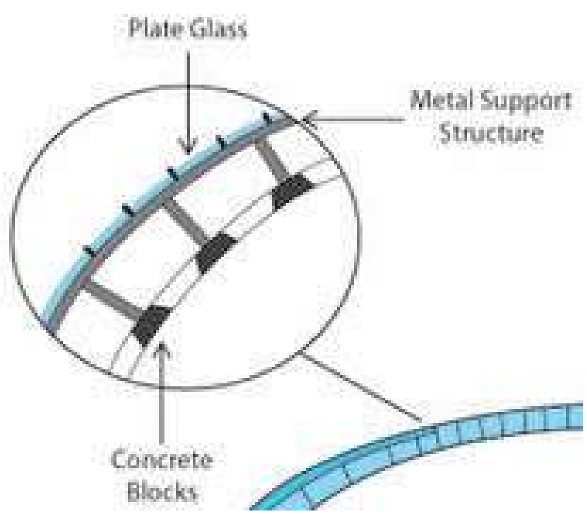




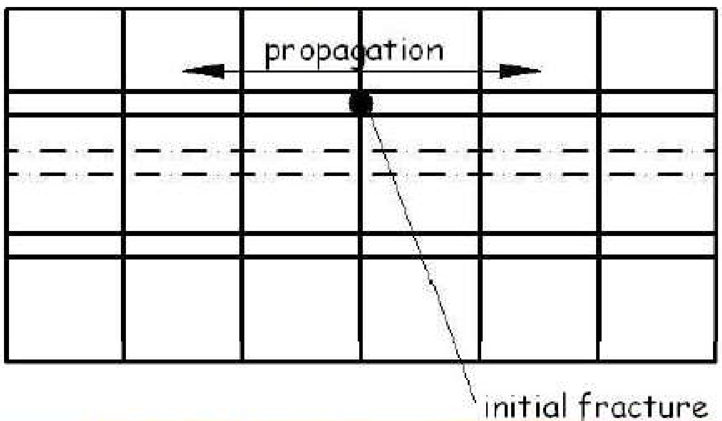

121.925
Photo Copyright © William Ronciere
AIRLINERS.NET

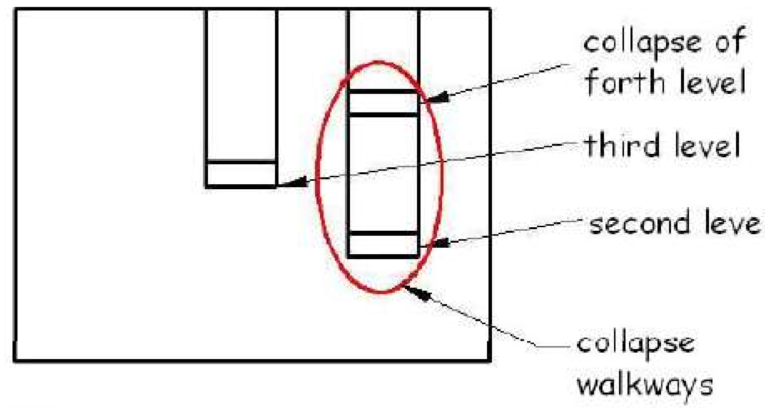
Causes of the Failure

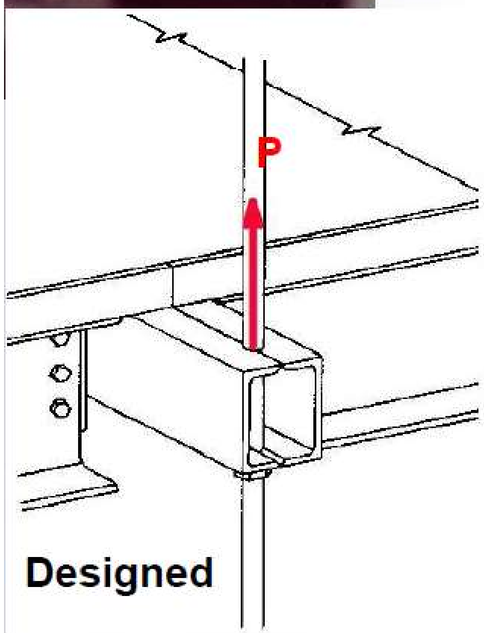
- *The metal support structure of the shell was found to be too deeply embedded into the concrete blocks*
- *This most likely caused cracking in the concrete layer/blocks, which led to the weakening of the roof, which then decreased the stability of the structure.*
- *The concrete supports/blocks, in many reports, was also considered to be insufficiently reinforced during pre-fabrication or the reinforcements could have been badly positioned during construction.*
- *"The horizontal concrete beams on which the shell rested were weakened by the passage of ventilation ducts"(Downey).*
- *Finally, one of the biggest factors that led to the collapse was the fact that rapid thermal expansion happened upon the outer metal structure, made the metal support structure to contract and expand the concrete.*



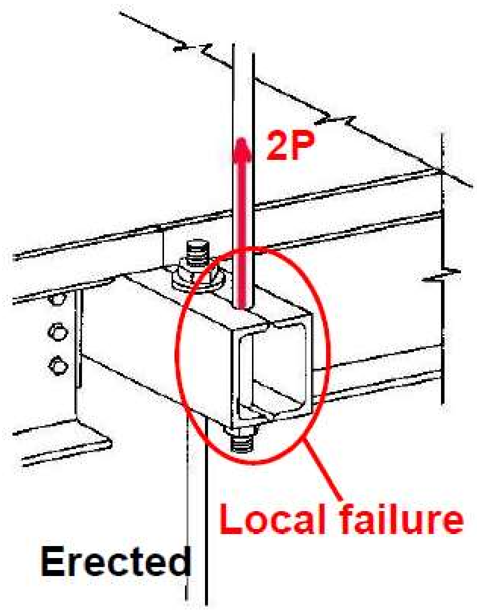
Collapse of Hyatt Regenary (Kansas City) walkways, 1981
due to modification of suspension details







Changing of designed solution.



Skyline Plaza, Bailey's Crossroads, Virginia

The collapse occurred because of the premature removal of shoring from beneath newly poured floors

Collapse of New World Hotel, Singapore, 1986

- The failure of three supporting column caused a fully progressive collapse
- The cause – additional or wrongly considered loads:
    - the building was designed without considering the dead load
    - Additional live loads during the service life .

Global approach to reduce the risk in case of extreme events

- Structural systems should be designed to be robust so as to avoid extensive damage (that may lead to the progressive collapse) under extreme events
- The basic strategies to reduce the probability of structural collapse may be expressed using the following equation:

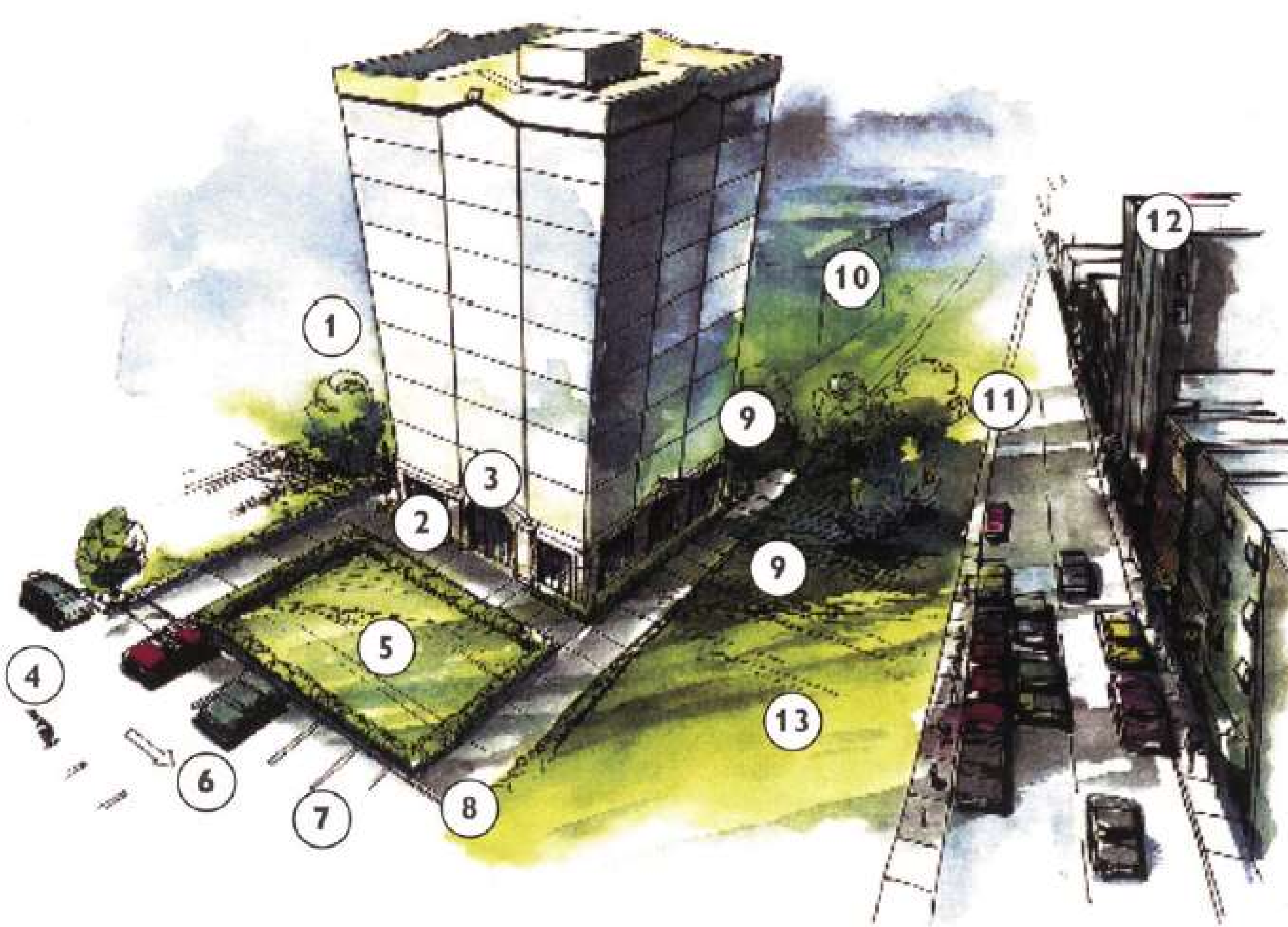
$$\boxed{P(C) = P(C|LD)\, P(LD|H)\, \lambda_H}$$

where:
λ_H = rate of occurrence of the abnormal load or hazard,
$P(LD|H)$ = probability of local damage given that the abnormal load occurs, and
$P(C|LD)$ = probability of collapse given that local damage occurs.

Reduce the probability of the abnormal load or hazard

- Outside the site boundary or defended perimeter
- In many cases, the easiest and cheapest method of securing protection is to adopt preventive measures:
  - External layout planning
  - Access control (security, closed circuit television)
  - Management procedures
- However, these measures may not be very effective, eg. important buildings must be located in cities and this limits the influence over the external layout. Also, building security design might have detrimental effects on the aesthetic and functional quality of buildings and their surroundings

Exemple of External layout planning



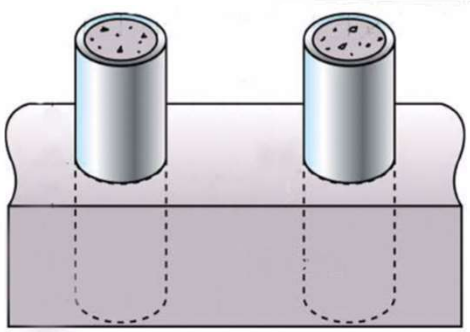
1. Locate assets stored on site, but outside the building within view of occupied rooms in the facility.
2. Eliminate parking beneath buildings.
3. Minimize exterior signage or other indications of asset locations.
4. Locate trash receptacles as far from the building as possible.
5. Eliminate lines of approach perpendicular to the building.
6. Locate parking to obtain stand-off distance from the building.
7. Illuminate building exteriors or sites where exposed assets are located.
8. Minimize vehicle access points.
9. Eliminate potential hiding places near the building; provide an unobstructed view around building.
10. Site building within view of other occupied buildings on the site.
11. Maximize distance from the building to the site boundary.
12. Locate building away from natural or manmade vantage points.
13. Secure access to power/heat plants, gas mains, water supplies, and electrical service.

Reinforced concrete barrier wall with artwork at the Scottish Parliament, Edinburgh

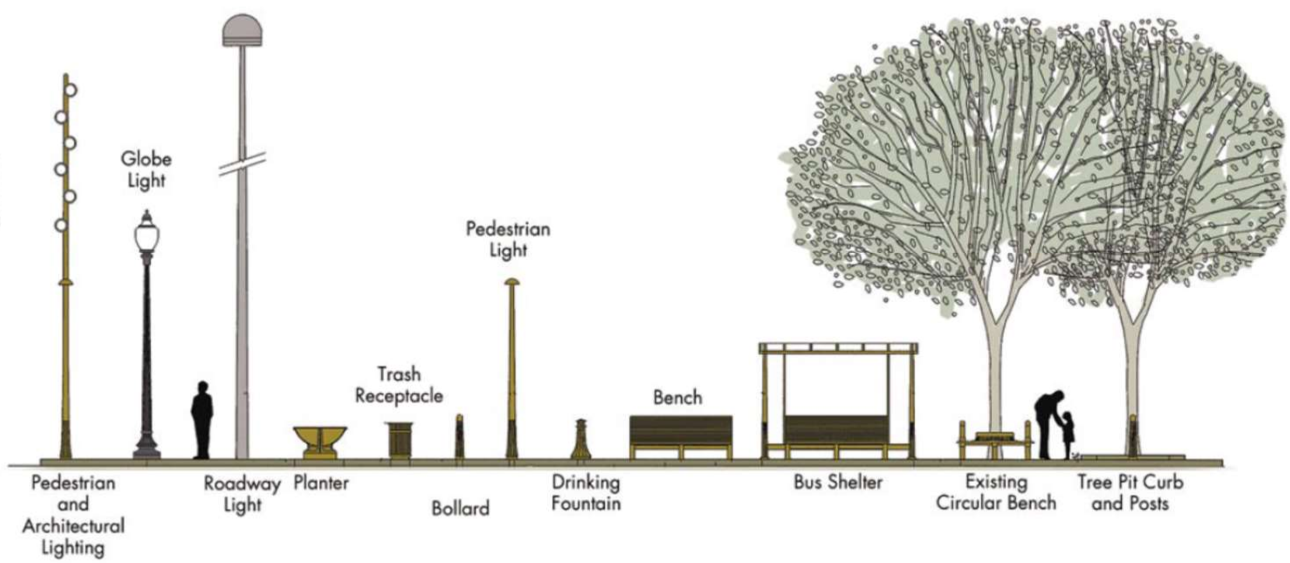
Risks of impact (with or without explosive)

- The impact of a plane is not considered (the airspace is under control)
- Impact of a truck has a limited probability of occurrence due to the localization of the embassy and of the controls at the entrances of the green zone
- The impact of a lightweight vehicle could be considered in case of problem with the safety system

anti-ram bollards/barriers



Urban design elements present opportunities to provide security



Risks of blast

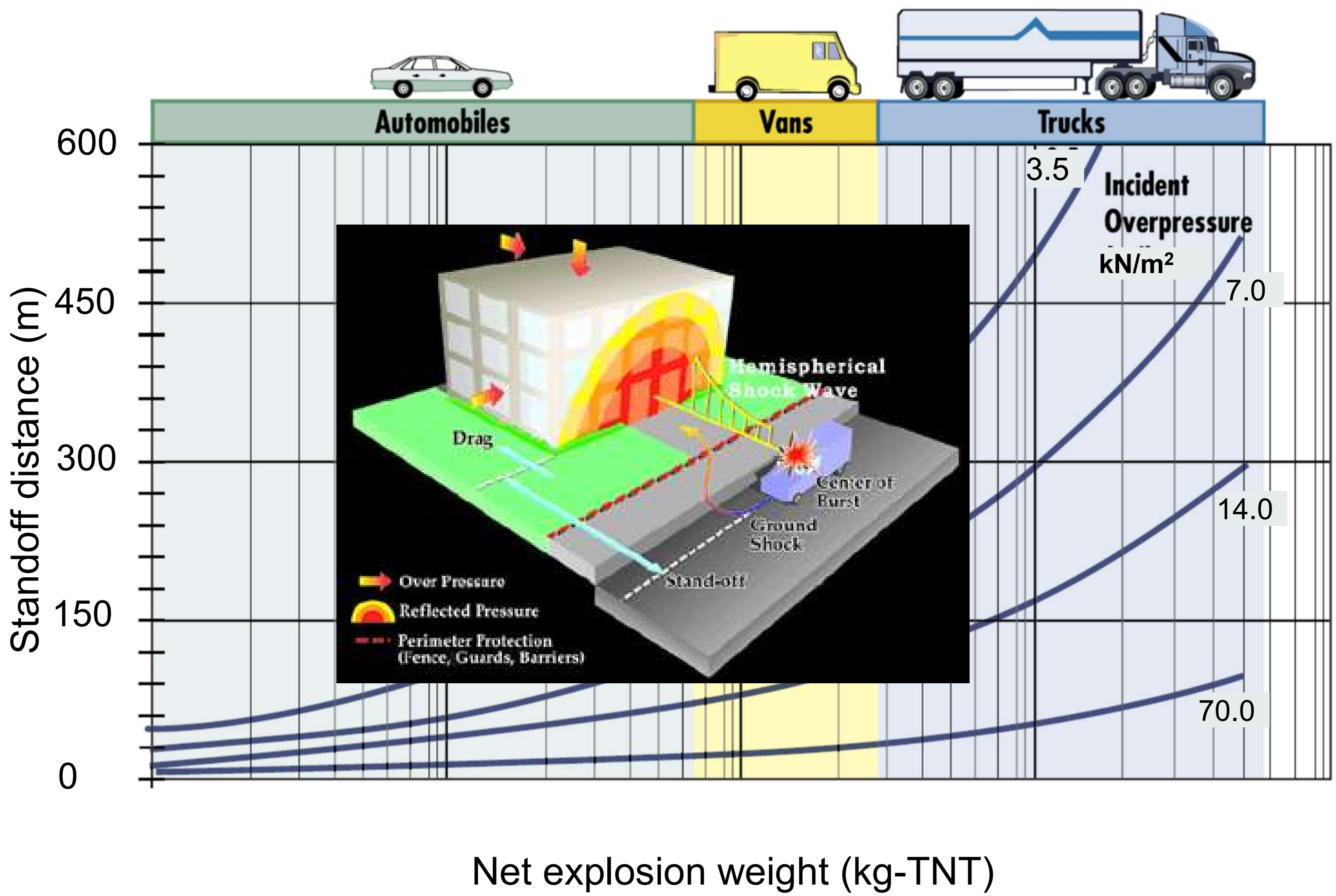
- Explosion of a vehicle parked close to the asset can be considered - anti-blast walls can be placed around the perimeter

- Suitcase with explosive close to the building - can be considered, but the explosion would be limited

Bombings of the US Embassies in Nairobi, Kenya, on August 7, 1998

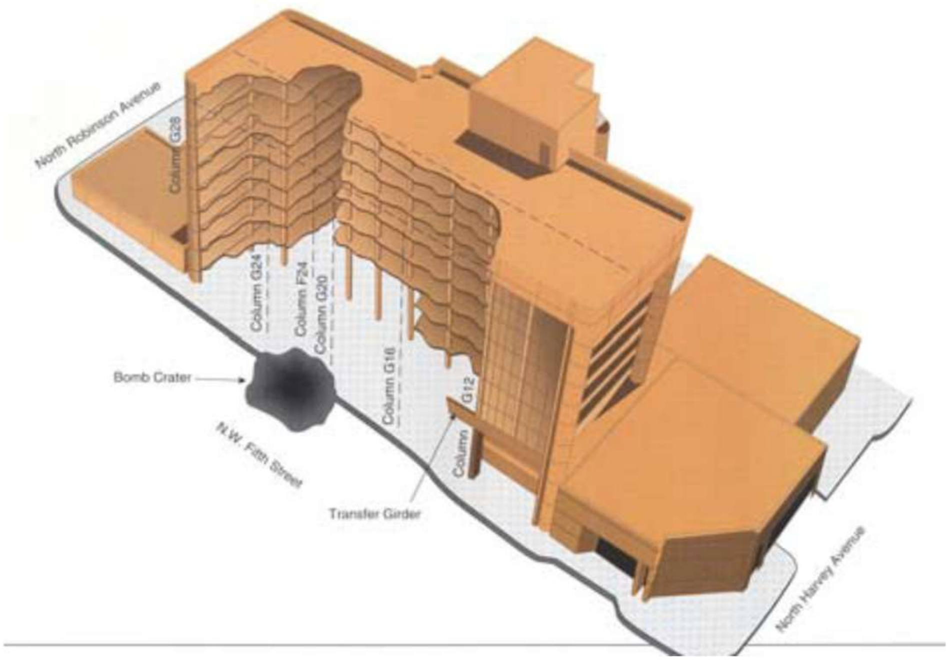
- *Six months before the attack, a report by State Department revealed embassy's extreme vulnerability due to lack of standoff (min. 100 ft standoff)*

Attacks on embassies: An aerial view shows the damage after the American embassy in Nairobi was bombed in 1998



Vehicle Bomb Sizes, Standoffs and Overpressures

- The building setback is a fundamental requirement of design
- In many cases not meeting this setback will translate into more severe design requirements for the exterior envelope



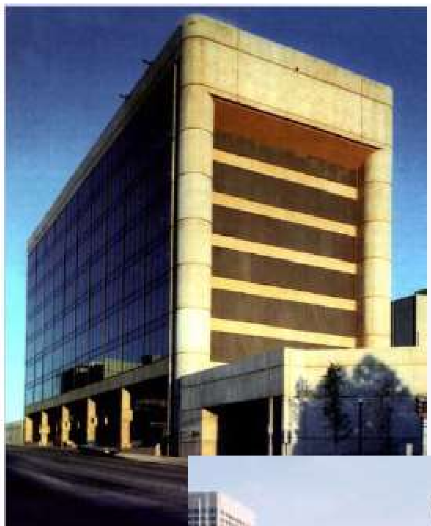
Alfred P. Murrah
Building,Oklahoma City

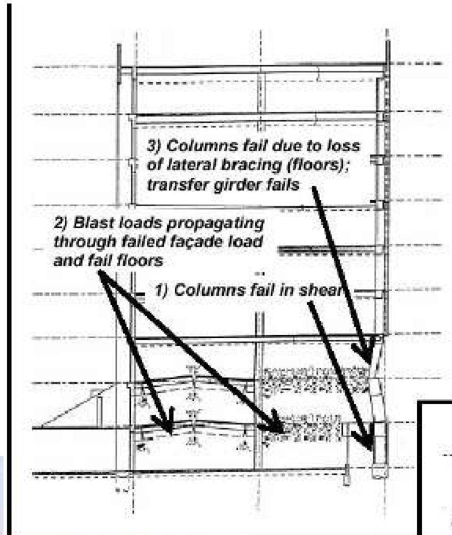
Stand-off distance: 1.5m

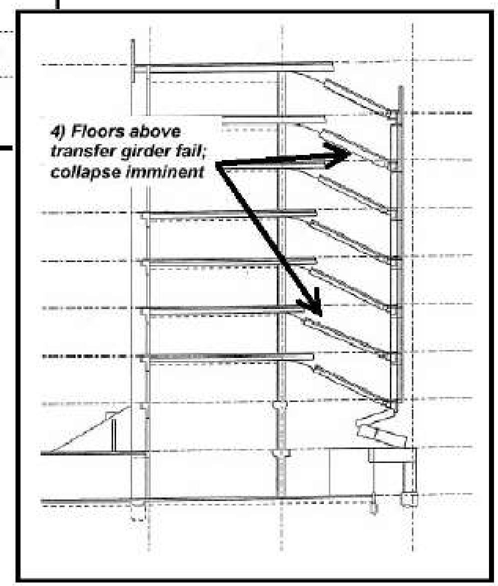
Equivalent TNT weight: 1814 kg

Blast

Murrah Federal Building, Oklahoma City, 1995

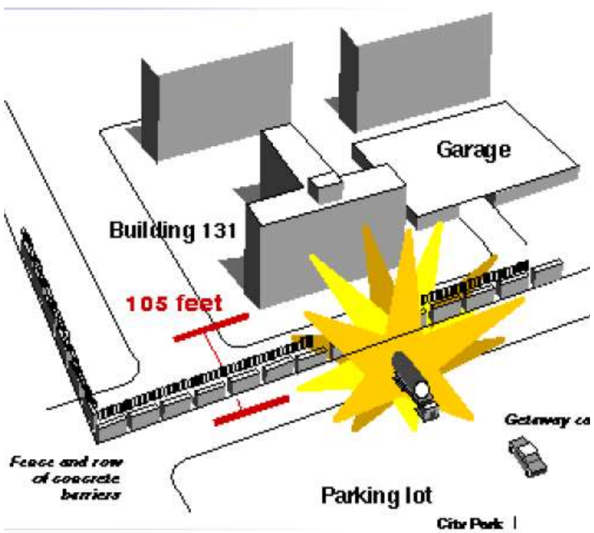






KHOBAR TOWERS – SAUDI ARABIA, 1996

place of explosion



LONDON 1992

LONDON 1993

IMPACT+EXPLOSION+FIRE

WORLD TRADE CENTER PROGRESSIVE COLLAPSE
11September 2001
New - York

Robustness measures can be categorized into ([1]):

- Local measures: focus on local values reaching critical levels when a member is lost, e.g.:
    1. the force demand-to-capacity ratio exceeds a threshold at a particular location
    2. the displacement or rotation at a given point exceed some prescribed limits.
- Global measures: are more comprehensive in their assessment, e.g.:
    1. pushdown methods, in which robustness is expressed as a ratio of the load carried by the damaged structure to the nominal gravity loads
    2. energy-based methods in which the vulnerability of the structure is assessed in terms of its ability to absorb energy before collapse after member loss.
    3. other global measures can be proposed, e.g., a redundancy measure, in which the number of adjacent members that must be removed to precipitate collapse is an indirect measure of overall robustness.

(1) El-Tawil, S. et al., 2013, Computational Simulation of Gravity-Induced Progressive Collapse of Steel-Frame Buildings: Current Trends and Future Research Needs, Journal of Structural Engineering.

The force demand-to-capacity ratio approach

- The demand-capacity ratio (*DCR*) may be defined as:

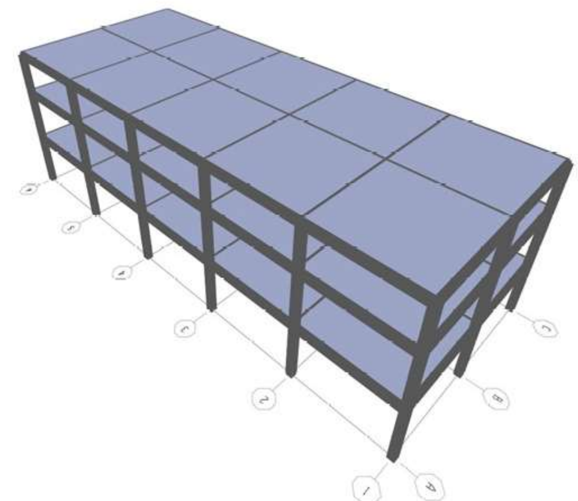
$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

where
Q_{UD} = acting force on structural member or joint, and
Q_{CE} = expected ultimate, unfactored capacity

- Using static, linear-elastic analysis, the designer identifies the magnitude and distribution of potential, inelastic demands on primary and secondary structural elements.
- The Design Guidelines limit the values of *DCR (*2 or less for typical structural configurations, and to 1.5 or less for atypical structural configurations.
- If the *DCR* cannot be limited to these values, then the structural member or connection in question is considered to have failed

Example

- Two spans and five bays of 6.0m each

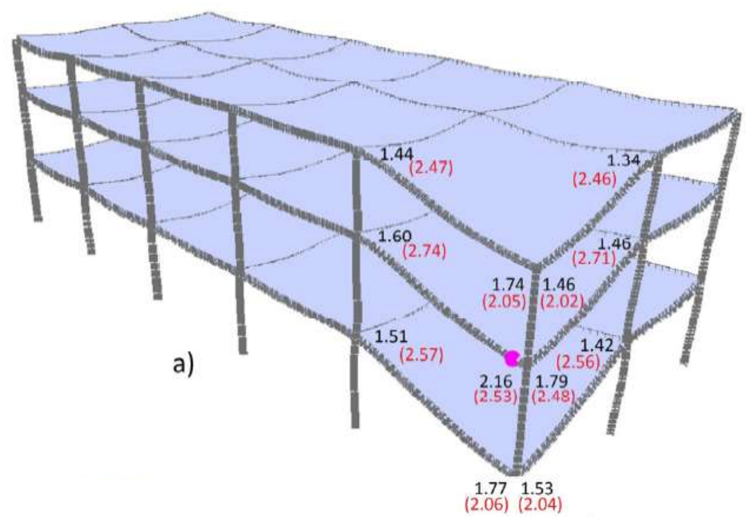


Structure	Column [mm]	Beam [mm]	Slab [mm]
3 -story	400 x 400	250 x 450	150

- Linear Static Analysis

$$Load = 2(DL + 0.25LL)$$

- SAP2000 structural analysis software



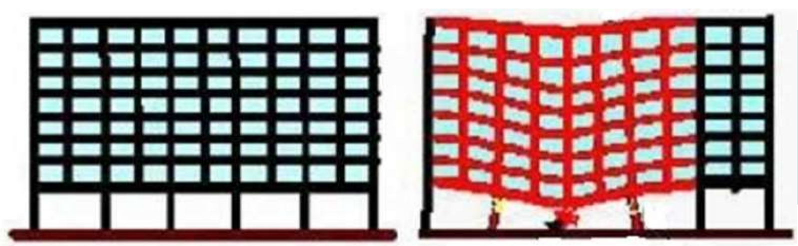
a)

Other measures to enhance robustness

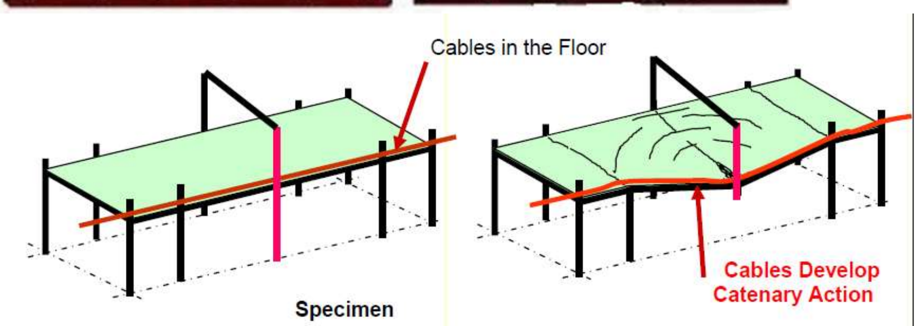
- Prediction of progressive collapse (worst case scenarios)
- Enhancing redundancy (to ensure that alternate load paths are available in the event of local failure of structural):
    - structure framing (two-way redundancy)
    - catenary action of floor (may be improved by using cables)
    - introduction of secondary trusses
    - relying on Vierendeel action
    - creation of “strong floors” in buildings
    - introduction of means to hang portions of the structure from above

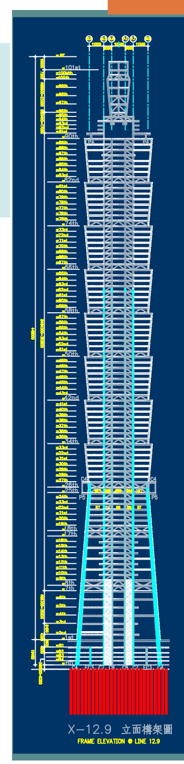
THE TAIPEI 101

10 very strong mfloors



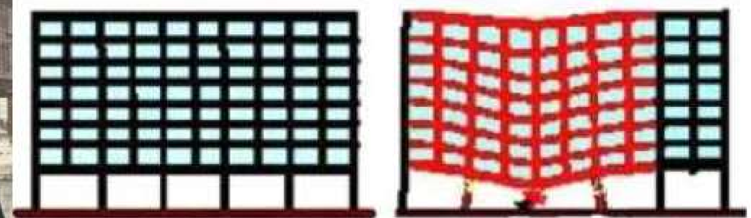
Catenary action of floor may be improved by using cables

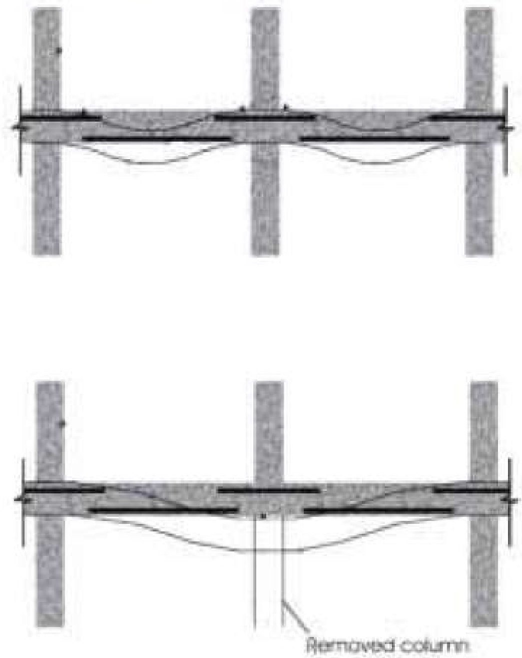




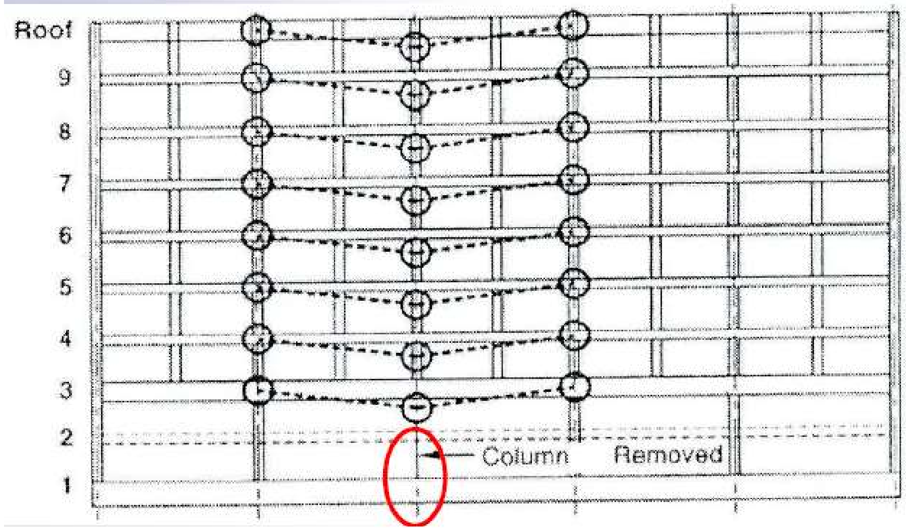
Bombing

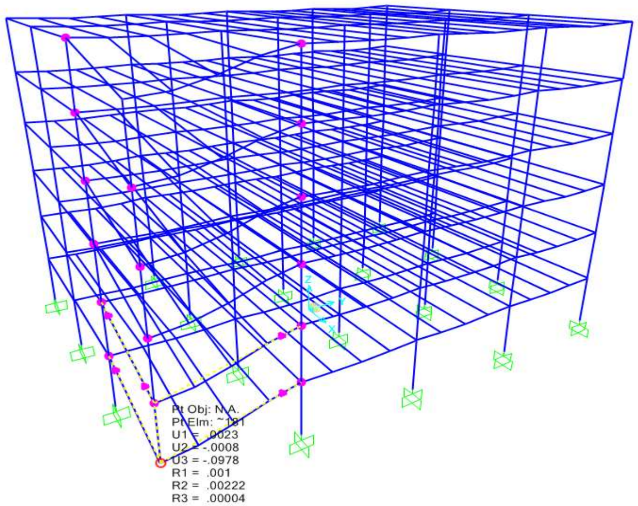
Catenary effect



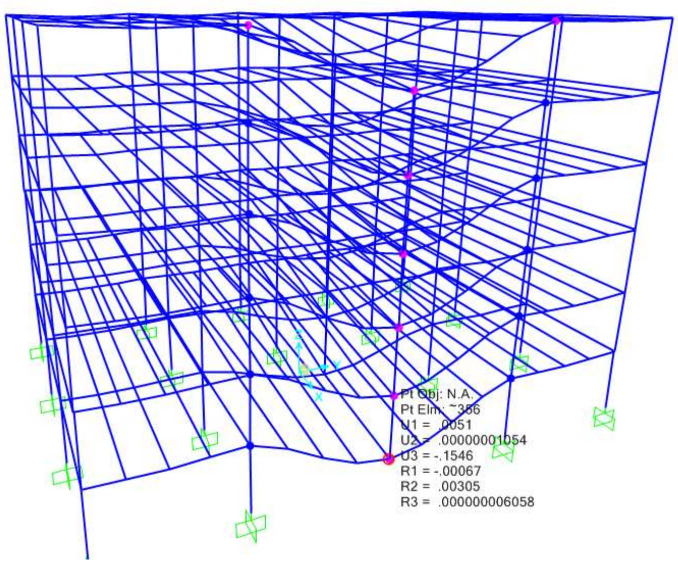


Collapse mechanism



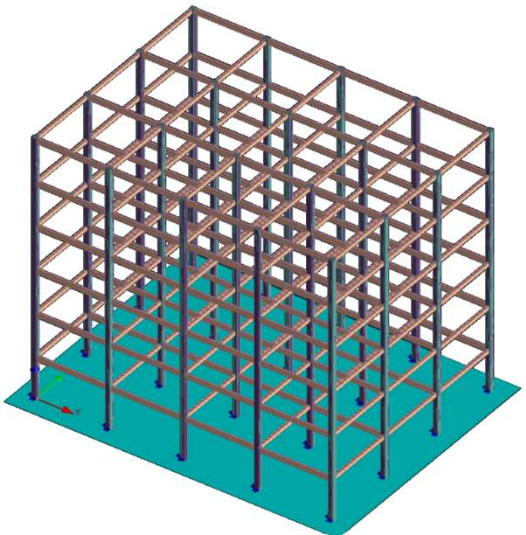


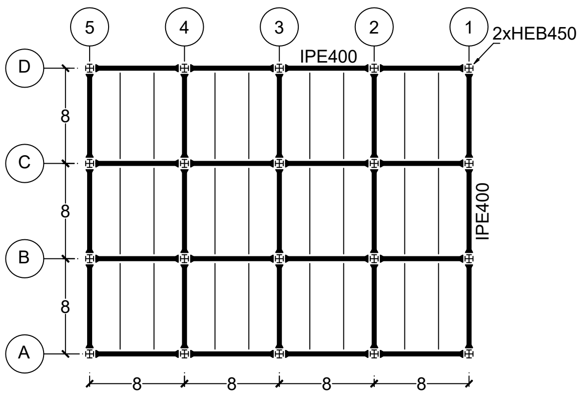
Scenario	Vertical displacement [mm]	Rotation [rad]
Corner column	97.8	0.00395

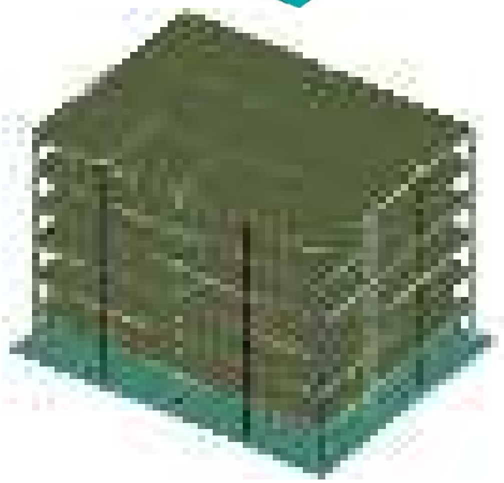


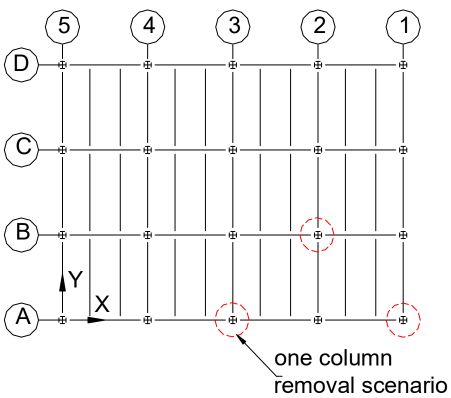
Scenario	Vertical displacement [mm]	Rotation [rad]
Perimeter + internal column	183	0.015

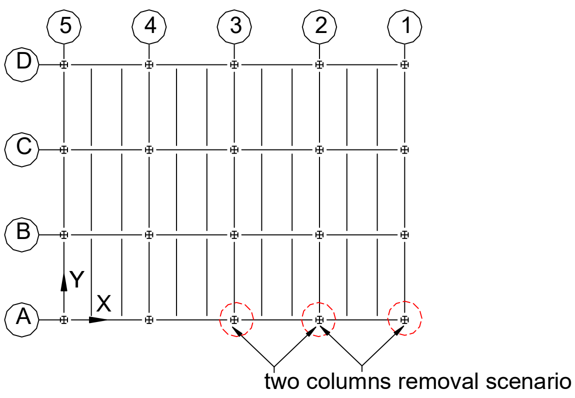
Pushdown methods: robustness is expressed as a ratio of the load carried by the damaged structure to the nominal gravity loads

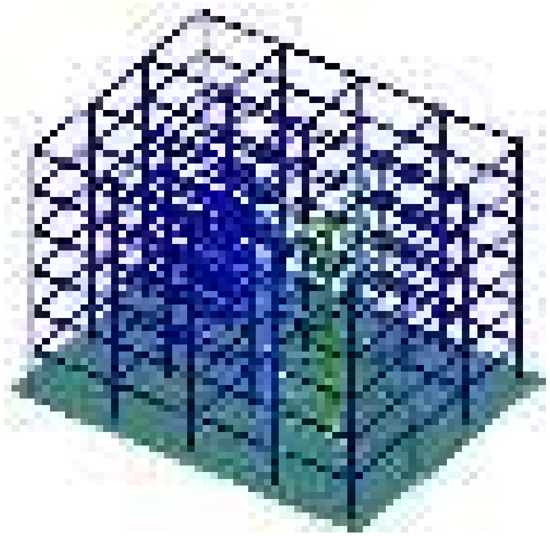


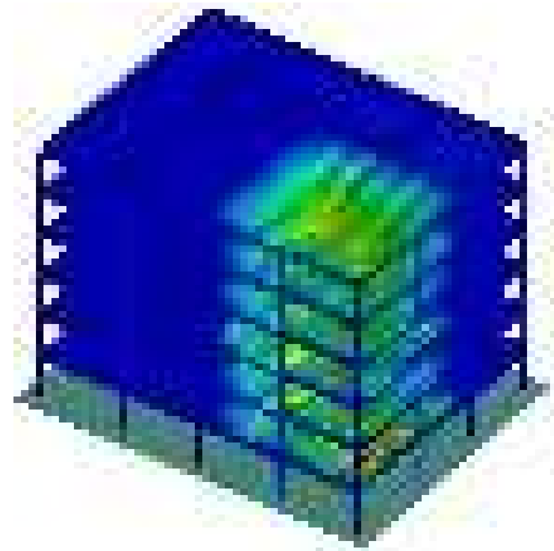


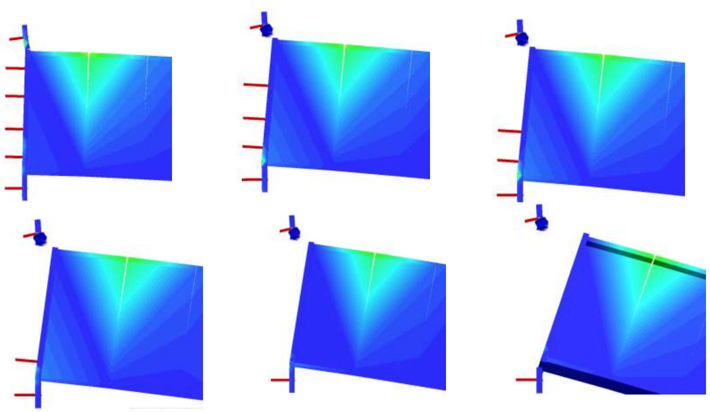


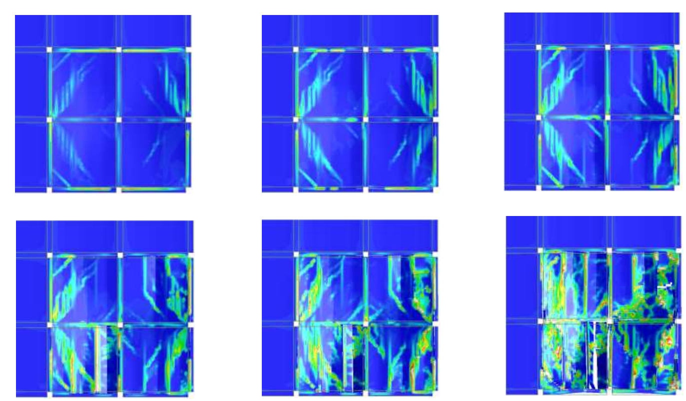




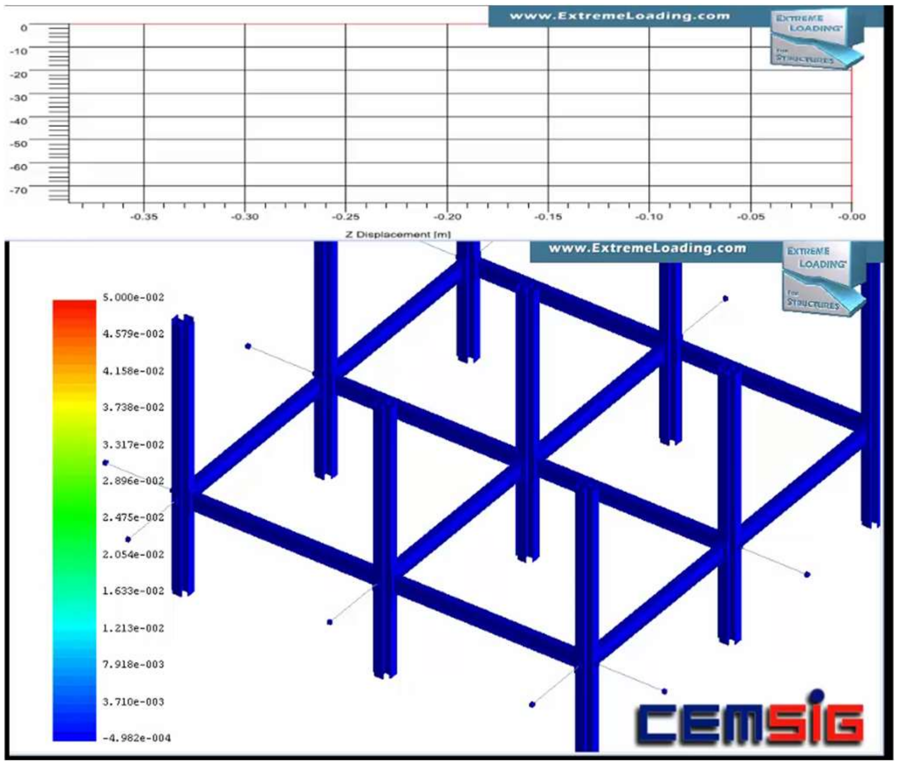








Scenario	Overload factor, Ω
	Dynamic analysis, Ω_D
S-I-A1	2.3
S-I-A3	1.8
S-I-B2	1.2
S-I-A12	1.2
S-I-A23	1.15
C-I-A1	2.83
C-I-A3	2.83
C-I-B2	2.91
C-I-A12	1.60
C-I-A23	1.94
S-II-A1	2.05
S-II-A3	1.6
S-II-B2	1.05
S-II-A12	1.1
S-II-A23	1.15
C-II-A1	2.66
C-II-A3	2.75
C-II-B2	2.58
C-II-A12	1.58
C-II-A23	1.91



www.ExtremeLoading.com
Extreme Loading for Structures
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-0.05
-0.00
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4.158e-002
3.738e-002
3.317e-002
2.896e-002
2.475e-002
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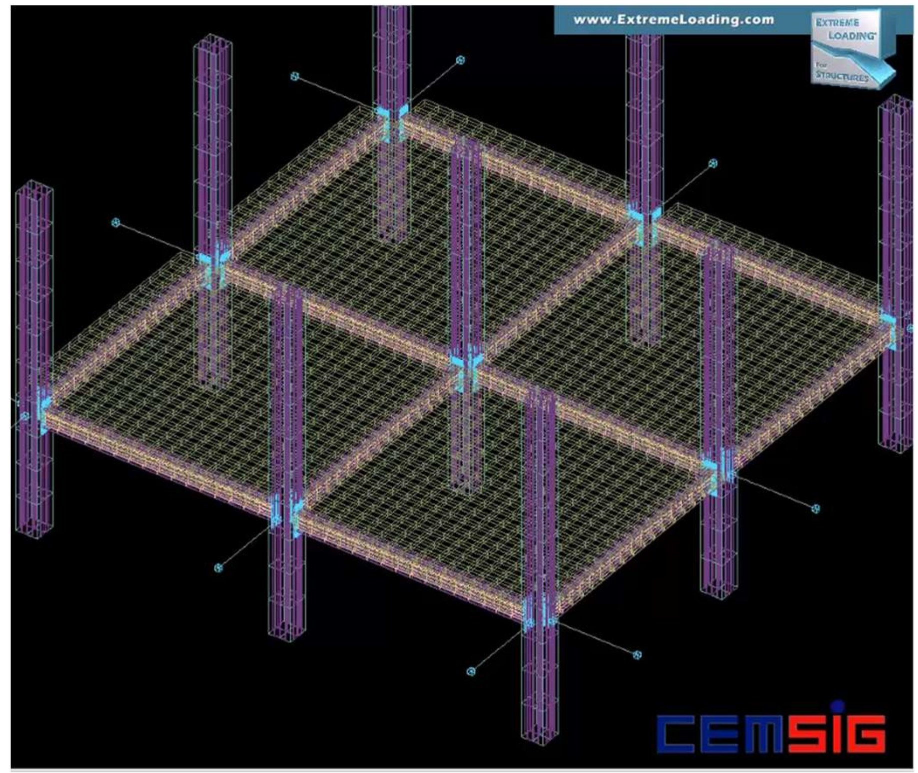
3.710e-003
-4.982e-004
CEMSIG



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Alternate load path method APM

(see UFC 4-023-03)

- The alternate load path method provides a formal check of the capability of the structural system to resist the removal of specific elements, such as a column at the building perimeter
- The load carried by the lost element must find an alternate load path to the building supports without initiating structural collapse.
- Large deformations are permitted before the onset of failure of an element.
- This method reduces the risk of progressive collapse by ensuring structural redundancy
- The method does not require characterization of the threat causing loss of the element, and is, therefore, a **threat independent approach**.

Advantages

- An advantage of this approach is that it promotes structural systems with ductility, continuity, and energy absorbing properties that are desirable in preventing progressive collapse.
- This method is also consistent with the seismic design approach:
  - **The seismic codes promote regular structures that are well tied together.**
  - **They also require ductile details so that plastic rotations can take place.**
- The removal locations are selected to verify that the structure has adequate flexural resistance to bridge over the missing element.

Types of analyses

- The transition from the original structural configuration to the damaged state is assumed to be instantaneous, exposing the structure to a dynamic effect.
- Can be used different analytical procedures:
  - **Elastic static**
  - **Inelastic static**
  - **Inelastic dynamic**
- Different accidental loads, lateral loads and combinations of loads for which the building stability should be checked

Combinations acc. to different codes

Standards	Load combinations after notional member removal	Accidental load
BS	$(1\pm0.5)\,D + L/3 + W_n/3$	34 kPa (5 psi)
Eurocode 2003 draft		20 kPa (3 psi)
Canada 1977	$D + L/3 + W_n/3$	
ASCE 7-98, 02, 05	$(0.9 \text{ or } 1.2)\,D + (0.5\,L \text{ or } 0.2\,S) + 0.2\,W_n$ (with member removal) $1.2\,D + A_k + (0.5\,L \text{ or } 0.2\,S)$ (specific local resistance method) $(0.9 \text{ or } 1.2)\,D + A_k + 0.2\,W_n$ (specific local resistance method)	A_k
DOD UFC 4-010-01	$D + 0.5\,L$ net floor uplift	
DOD UFC 4-023-03	$D + 0.5\,L$ net floor uplift $(0.9 \text{ or } 1.2)\,D + (0.5\,L \text{ or } 0.2\,S) + 0.2\,W_n$ (nonlinear dynamic analysis) $2.0\,[(0.9 \text{ or } 1.2)\,D + (0.5\,L \text{ or } 0.2\,S)] + 0.2\,W$ (static analysis)	
NYC 1998, 2003	$2\,D + 0.25\,L + 0.2\,W_n$	
GSA	$2\,(D + 0.25\,L)$ static analysis $D + 0.25\,L$ dynamic analysis	
Sweden	$G_k + \Psi Q_k$	Q_{ak}

D, L, W_n, S = dead, live, wind and snow loads;
Q_{ak} = characteristic value of accidental action;
G_k, Q_k = characteristic dead, imposed loads per unit area of the floor or roof; Ψ is a load reduction factor which, when multiplied with Q_k, gives the frequent value of a variable action.
A_k = extraordinary load.

The amplification of the gravity loads applies only to the section of the structure directly below the failed element!

Linear static procedure LSP

- the characteristic load is amplified with a dynamic load factor (most codes propose a value of two for the dynamic factor).
- elastic static methods may sometimes mask hazardous dynamic effects and should be limited to simple structures (Marjanishvili 2004).

Nonlinear static procedure NSP

- The non-linear equivalent static approaches generally simulate the dynamic load through a load factor.
- The gravity load reaction of the removed column are incrementally applied to generate a “push-down curve” of the structural behavior.
- Acceptance criteria for member performance are based on deformation limits

Nonlinear dynamic procedure NDP

- A more rigorous approach for evaluating progressive collapse
- This approach should be used by structural engineers with knowledge and experience in structural dynamics
- Acceptance criteria for the performance of structural members are in terms of deformation limits.

Load combinations

- Two load cases need to be applied and analyzed:
  - one for the deformation-controlled actions
  - one for the force-controlled actions

Load Case for Nonlinear Static Procedure NSP

- **Increased Gravity Loads for Floor Areas Above Removed Column or Wall (see next figure)**.

Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element

GN = ΩN [1.2 D + (0.5 L or 0.2 S)]

where ***GN*** = Increased gravity loads for Nonlinear Static Analysis

D = Dead load including façade loads (kN/m2)

L = Live load (kN/m2)

S = Snow load (kN/m2)

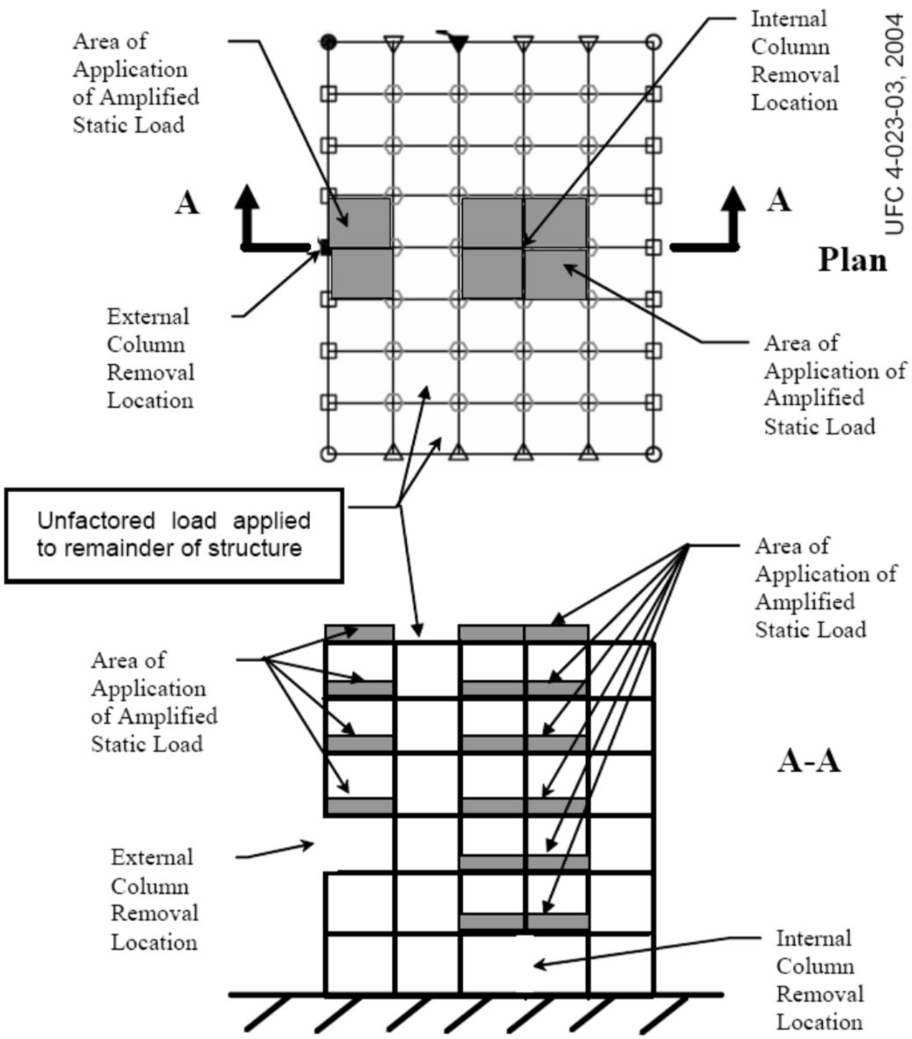
ΩN = Dynamic increase factor for calculating deformation-controlled and force-controlled actions for Nonlinear Static analysis;

- **Gravity Loads for Floor Areas Away From Removed Column or Wall (see next figure)**.

Apply the following gravity load combination to those bays not loaded with ***GN***:

G = 1.2 D + (0.5 L or 0.2 S)

where ***G*** = Gravity loads



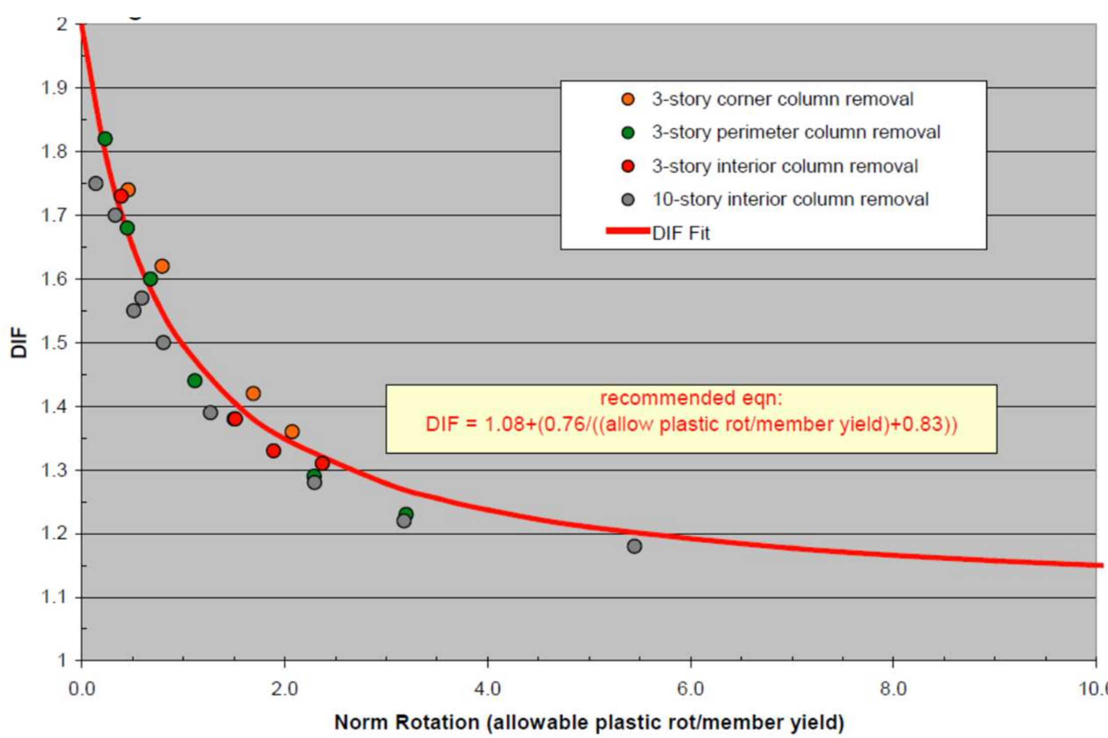
Load Application for Alternate Path Analysis (UFC 4-023-03)

Dynamic Increase Factors for Nonlinear Static Analysis

Material	Structure Type	Ω_N
Steel	Framed	$1.08 + 0.76/(\theta_{pra}/\theta_y + 0.83)$
Reinforced Concrete	Framed	$1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$
	Load-Bearing Wall	2
Masonry	Load-bearing Wall	2
Wood	Load-bearing Wall	2
Cold-formed Steel	Load-bearing Wall	2

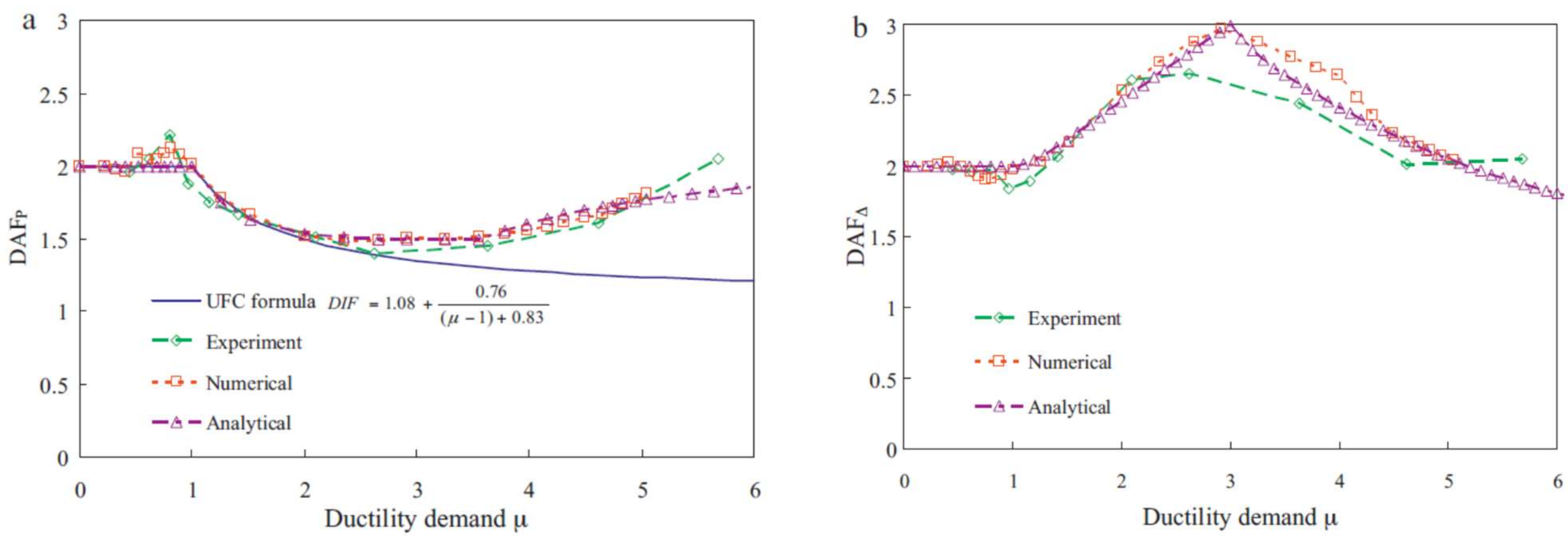
Dynamic load factor

- For both Linear Static and Nonlinear Static, the UFC 4-023-03 and the GSA Guidelines use a load multiplier (conservative DIF = 2), applied directly to the progressive collapse load combination.



Dynamic Increase Factor for Structural Steel

Tsai & You, 2012



a) Comparison of predicted force-based DAFs; (b) Comparison of predicted displacement-based DAFs

Load Case for Nonlinear Dynamic Procedure NDP

- **Gravity Loads for Entire Structure**.

Apply the following gravity load combination to the entire structure:

$\boldsymbol{G_{ND} = 1.2\,D + (0.5\,L \text{ or } 0.2\,S)}$

where G_{ND} = Gravity loads for Nonlinear Dynamic Analysis

D = Dead load including façade loads (kN/m2)

L = Live load (kN/m2)

S = Snow load (kN/m2)

Loading Procedure

- Starting at zero load, monotonically and proportionately increase the gravity loads to the entire model (i.e., the column or wall section have not been removed yet) until equilibrium is reached.
- After equilibrium is reached for the framed and load-bearing wall structures, remove the column or wall section.
  - it is preferable to remove the column or wall section instantaneously
  - otherwise, the duration for removal must be less than one tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column, as determined from the analytical model with the column or wall section removed.
- The analysis shall continue until the maximum displacement is reached or one cycle of vertical motion occurs at the column or wall section removal location.

Modelling parameters

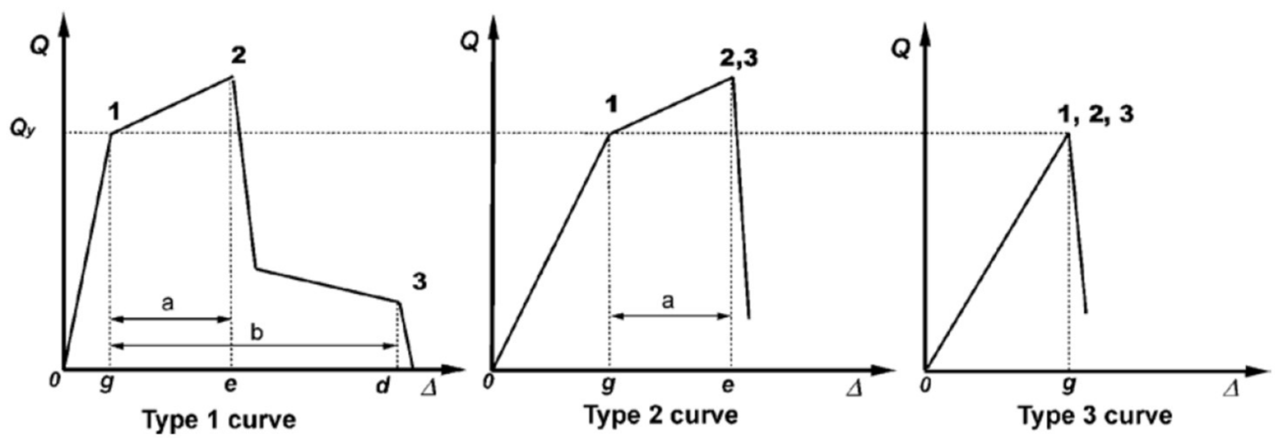
- Performance criteria for structure response to column loss (folllowing blast, impact) are component dependant: beams, columns, connections plates have distinct criteria
- Limitation of displacement or deformation demands:
  - The deflection and deformations that are calculated must be compared against the deformation limits that are specific to each structural component
  - If any structural element or connection violates an acceptability criteria, modifications must be made to the structure

Primary and Secondary Components

- Classify structural elements and components as either primary or secondary
- Structural elements and components that provide the capacity of the structure to resist collapse due to removal of a vertical load-bearing element as primary.
- Classify all other elements and components as secondary.
- For example, a steel gravity beam may be classified as secondary if it is assumed to be pinned at both ends to girders and the designer chooses to ignore any flexural strength at the connection; if the connection is modeled as partially restrained and thus contributes to the resistance of collapse, it is a primary member.

Force- and Deformation-Controlled Actions

- Classify all actions as either deformation-controlled or force-controlled using the component force versus deformation curve
- Define a primary component action as deformation-controlled if it has a Type 1 curve and e ≥ 2g, or, it has a Type 2 curve and e ≥ 2g.
- Define a primary component action as force-controlled if it has a Type 1 or Type 2 curve and e < 2g, or, if it has a Type 3 curve.
- Define a secondary component action as deformation-controlled if it has a Type 1 curve for any e/g ratio or if it has a Type 2 curve and e ≥ 2g.
- Define a secondary component action as force controlled if it has a Type 2 curve and e < 2g, or, if it has a Type 3 curve.



Definition of Force-Controlled and Deformation-Controlled Actions, from ASCE 41

Examples of Deformation-Controlled and Force-Controlled Actions, from ASCE 41

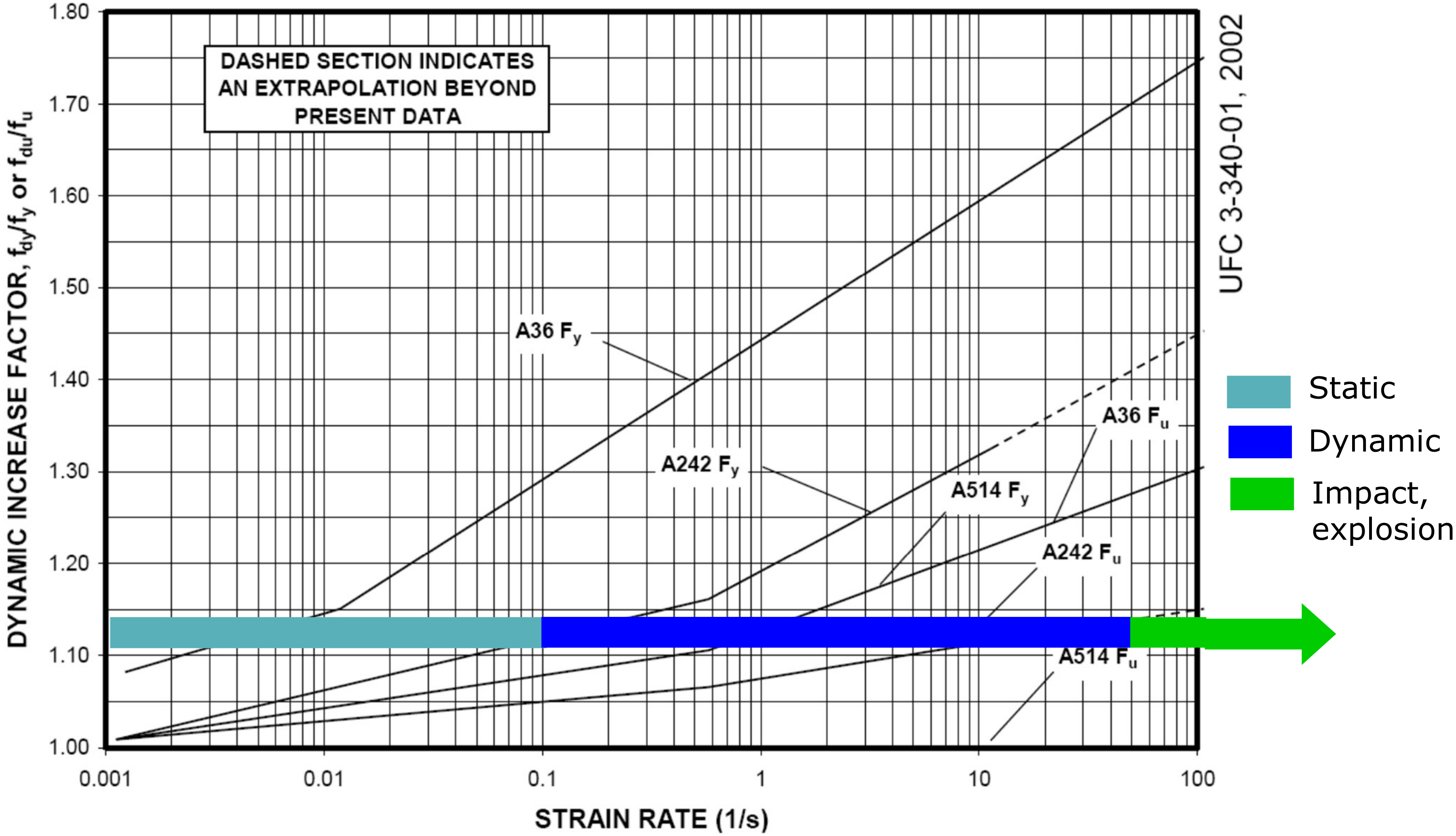
Component	Deformation-Controlled Action	Force- Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M[2]	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.
2. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

Material Properties

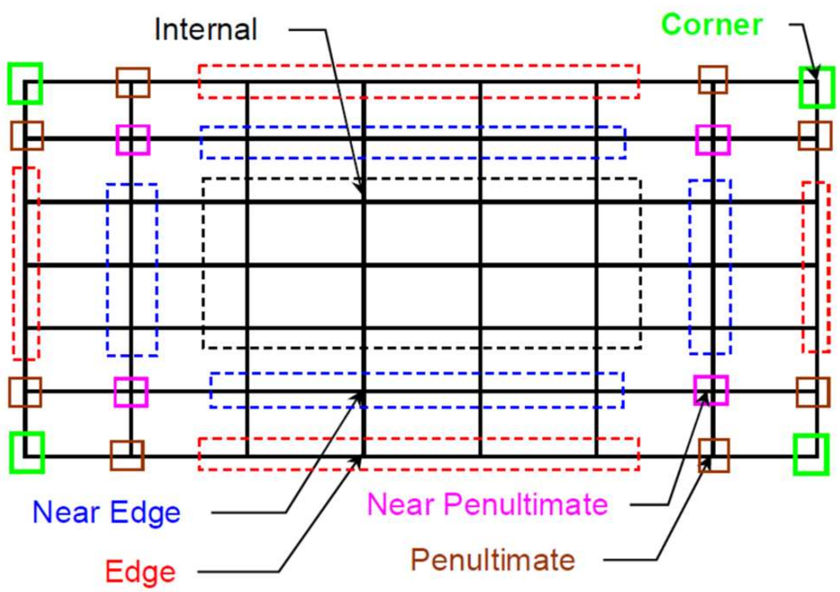
- Expected material properties such as yield strength, ultimate strength, weld strength, fracture toughness, elongation, etc, shall be based on mean values of tested material properties.
- Lower bound material properties shall be based on mean values of tested material properties minus one standard deviation.
- Ductility and strain rate sensitivity are key parameters for the performance under dynamic loading

Strain Rate Factors for Structural Steel



Location defined for element removal

- Load-bearing elements are removed at different locations
- For each location, analyses are done for the following stories:
  - First story above grade
  - Story directly below roof
  - Story at mid-height
  - Story above the location of a change in wall size



Location of Internal/external Column Removal

Acceptance Criteria

UFC Criteria, 2016

- With a few notable exceptions, the acceptance criteria for linear and nonlinear approaches and the modeling criteria for nonlinear approaches from ASCE 41 are employed in the updated UFC 4-023-03 (2016).
- The ASCE 41 criteria are considered to be conservative when applied to progressive collapse design as they have been developed for repeated load cycles (i.e., backbone curves) whereas only one half load cycle is applied in progressive collapse.
- Since 2009 edition, adoption of modeling parameters and acceptance criteria from ASCE 41 Seismic Rehabilitation of Existing Buildings

Component	AP for Low LOP		AP for Medium and High LOP	
	Ductility (μ)	Rotation, Degrees (θ)	Ductility (μ)	Rotation, Degrees (θ)
Beams--Seismic Section[A]	20	12	10	6
Beams--Compact Section[A]	5	-	3	-
Beams--Non-Compact Section[A]	1.2	-	1	-
Plates	40	12	20	6
Columns and Beam-Columns	3	-	2	-
Steel Frame Connections; Fully Restrained				
Welded Beam Flange or Coverplated (all types)[B]	-	2.0	-	1.5
Reduced Beam Section[B]	-	2.6	-	2
Steel Frame Connections; Partially Restrained				
Limit State governed by rivet shear or flexural yielding of plate, angle or T-section[B]	-	2.0	-	1.5
Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section[B]	-	1.3	-	0.9

Acceptability Criteria and Deformation Limits for Steel Members (UFC Criteria, 2005)

Table 5-6. Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components

	Modeling Parameters			Acceptance Criteria[14]				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	a	b	c	IO	LS	CP	LS	CP
Beams—Flexure								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
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 lower resulting value shall be used							
Columns—Flexure[2,7]								
For $P/P_{CL} < 0.2$								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
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 lower resulting value shall be used							
For $0.2 \leq P/P_{CL} \leq 0.5$								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{260}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	$0.25\theta_y$	—[5]	—[3]	—[6]	—[4]
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{400}{\sqrt{F_{ye}}}$	$1\theta_y$	$1.5\theta_y$	0.2	$0.25\theta_y$	$0.5\theta_y$	$0.8\theta_y$	$1.2\theta_y$	$1.2\theta_y$
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 lower resulting value shall be used							

Column Panel Zones	$12\theta_y$	$12\theta_y$	1.0	$1\theta_y$	$8\theta_y$	$11\theta_y$	$12\theta_y$	$12\theta_y$
Fully Restrained Moment Connections[13]								
WUF[12]	$0.051 - 0.0013d$	$0.043 - 0.00060d$	0.2	$0.026 - 0.00065d$	$0.0337 - 0.00086d$	$0.0284 - 0.00040d$	$0.0323 - 0.00045d$	$0.043 - 0.00060d$
Bottom Haunch in WUF with Slab	0.026	0.036	0.2	0.013	0.0172	0.0238	0.0270	0.036
Bottom Haunch in WUF without Slab	0.018	0.023	0.2	0.009	0.0119	0.0152	0.0180	0.023
Welded Cover Plate in WUF[12]	$0.056 - 0.0011d$	$0.056 - 0.0011d$	0.2	$0.028 - 0.00055d$	$0.0319 - 0.00063d$	$0.0426 - 0.00084d$	$0.0420 - 0.00083d$	$0.056 - 0.0011d$
Improved WUF—Bolted Web[12]	$0.021 - 0.00030d$	$0.050 - 0.00060d$	0.2	$0.010 - 0.00015d$	$0.0139 - 0.00020d$	$0.0210 - 0.00030d$	$0.0375 - 0.00045d$	$0.050 - 0.00060d$
Improved WUF—Welded Web	0.041	0.054	0.2	0.020	0.0312	0.0410	0.0410	0.054
Free Flange[12]	$0.067 - 0.0012d$	$0.094 - 0.0016d$	0.2	$0.034 - 0.00060d$	$0.0509 - 0.00091d$	$0.0670 - 0.0012d$	$0.0705 - 0.0012d$	$0.094 - 0.0016d$
Reduced Beam Section[12]	$0.050 - 0.00030d$	$0.070 - 0.00030d$	0.2	$0.025 - 0.00015d$	$0.0380 - 0.00023d$	$0.0500 - 0.00030d$	$0.0525 - 0.00023d$	$0.07 - 0.00030d$
Welded Flange Plates								
a. Flange Plate Net Section	0.03	0.06	0.2	0.015	0.0228	0.0300	0.0450	0.06
b. Other Limit States	Force-controlled							
Welded Bottom Haunch	0.027	0.047	0.2	0.014	0.0205	0.0270	0.0353	0.047

	Modeling Parameters			Acceptance Criteria[14]				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	*a*	*b*	*c*	IO	LS	CP	LS	CP
Welded Top and Bottom Haunches	0.028	0.048	0.2	0.014	0.0213	0.0280	0.0360	0.048
Welded Cover—Plated Flanges	0.031	0.031	0.2	0.016	0.0177	0.0236	0.0233	0.031
Partially Restrained Moment Connections								
Top and Bottom Clip Angle[9]								
a. Shear Failure of Rivet or Bolt (Limit State 1)[8]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Tension Failure of 0.012 Horizontal Leg of Angle (Limit State 2)	0.018	0.800	0.003	0.008	0.010	0.010	0.015	
c. Tension Failure of Rivet or Bolt (Limit State 3)[8]	0.016	0.025	1.000	0.005	0.008	0.013	0.020	0.020
d. Flexural Failure of Angle (Limit State 4)	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
Double Split Tee[9]								
a. Shear Failure of Rivet or Bolt (Limit State 1)[8]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Tension Failure of Rivet or Bolt (Limit State 2)[8]	0.016	0.024	0.800	0.005	0.008	0.013	0.020	0.020
c. Tension Failure of Split Tee Stem (Limit State 3)	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
d. Flexural Failure of Split Tee (Limit State 4)	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070

Bolted Flange Plate[9]								
a. Failure in Net Section of Flange Plate or Shear Failure of Bolts or Rivets[8]	0.030	0.030	0.800	0.008	0.020	0.025	0.020	0.025
b. Weld Failure or Tension Failure on Gross Section of Plate	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
Bolted End Plate								
a. Yield of End Plate	0.042	0.042	0.800	0.010	0.028	0.035	0.035	0.035
b. Yield of Bolts	0.018	0.024	0.800	0.008	0.010	0.015	0.020	0.020
c. Failure of Weld	0.012	0.018	0.800	0.003	0.008	0.010	0.015	0.015
Composite Top Clip Angle Bottom[9]								
a. Failure of Deck Reinforcement	0.018	0.035	0.800	0.005	0.010	0.015	0.020	0.030
b. Local Flange Yielding and Web Crippling of Column	0.036	0.042	0.400	0.008	0.020	0.030	0.025	0.035
c. Yield of Bottom Flange Angle	0.036	0.042	0.200	0.008	0.020	0.030	0.025	0.035
d. Tensile Yield of Rivets or Bolts at Column Flange	0.015	0.022	0.800	0.005	0.008	0.013	0.013	0.018
e. Shear Yield of Beam Flange Connection	0.022	0.027	0.200	0.005	0.013	0.018	0.018	0.023
Shear Connection with Slab[12]	$0.029-0.00020d_{bg}$	$0.15-0.0036d_{bg}$	0.400	$0.014-0.00010d_{bg}$	—	—	$0.1125-0.0027d_{bg}$	$0.15-0.0036d_{bg}$
Shear Connection without Slab[12]	$0.15-0.0036d_{bg}$	$0.15-0.0036d_{bg}$	0.400	$0.075-0.0018d_{bg}$	—	—	$0.1125-0.0027d_{bg}$	$0.15-0.0036d_{bg}$

Table 5-6. (Continued)

	Modeling Parameters			Acceptance Criteria[14]				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	a	b	c	IO	LS	CP	LS	CP
EBF Link Beam[10,11]								
a. $e \leq \frac{1.6\,M_{CE}}{V_{CE}}$	0.15	0.17	0.8	0.005	0.11	0.14	0.14	0.16
b. $e \geq \frac{2.6\,M_{CE}}{V_{CE}}$	Same as for beams							
c. $\frac{1.6\,M_{CE}}{V_{CE}} < e < \frac{2.6\,M_{CE}}{V_{CE}}$	Linear interpolation shall be used							
Steel Plate Shear Walls[1]	$14\theta_y$	$16\theta_y$	0.7	$0.5\theta_y$	$10\theta_y$	$13\theta_y$	$13\theta_y$	$15\theta_y$

[1]Values are for shear walls with stiffeners to prevent shear buckling.
[2]Columns in moment or braced frames shall be permitted to be designed for the maximum force delivered by connecting members. For rectangular or square columns, replace $b_f/2t_f$ with b/t, replace 52 with 110, and replace 65 with 190.
[3]Plastic rotation $= 11\,(1 - 5/3\,P/P_{CL})\,\theta_y$.
[4]Plastic rotation $= 17\,(1 - 5/3\,P/P_{CL})\,\theta_y$.
[5]Plastic rotation $= 8\,(1 - 5/3\,P/P_{CL})\,\theta_y$.
[6]Plastic rotation $= 14\,(1 - 5/3\,P/P_{CL})\,\theta_y$.
[7]Columns with $P/P_{CL} > 0.5$ shall be considered force-controlled.
[8]For high-strength bolts, divide values by 2.0.
[9]Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 18$ in., multiply m-factors by $18/d_b$.
[10]Deformation is the rotation angle between link and beam outside link or column.
[11]Values are for link beams with three or more web stiffeners. If no stiffeners, divide values by 2.0. Linear interpolation shall be used for one or two stiffeners.
[12]d is the beam depth; d_{bg} is the depth of the bolt group. Where plastic rotations are a function of d or d_{bg}, they need not be taken as less than 0.0.
[13]Tabulated values shall be modified as indicated in Section 5.4.2.4.3, Item 4.
[14]Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.

Table 5-7. Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components—Axial Actions

	Modeling Parameters			Acceptance Criteria[6]				
				Plastic Deformation				
	Plastic Deformation		Residual Strength Ratio		Primary		Secondary	
Component/Action	a	b	c	IO	LS	CP	LS	CP
Braces in Compression (except EBF braces)[1,2]								
a. Slender								
$\frac{Kl}{r} \geq 4.2\sqrt{E/F_y}$								
1. W, I, 2L In-Plane[3], 2C In-Plane[3]	$0.5\Delta_c$	$10\Delta_c$	0.3	$0.25\Delta_c$	$6\Delta_c$	$8\Delta_c$	$8\Delta_c$	$10\Delta_c$
2. 2L Out-of-Plane[3], 2C Out-of-Plane[3]	$0.5\Delta_c$	$9\Delta_c$	0.3	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$9\Delta_c$
3. HSS, Pipes, Tubes	$0.5\Delta_c$	$9\Delta_c$	0.3	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$9\Delta_c$
b. Stocky[4]								
$\frac{Kl}{r} \leq 2.1\sqrt{E/F_y}$								
1. W, I, 2L In-Plane[3], 2C In-Plane[3]	$1\Delta_c$	$8\Delta_c$	0.5	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
2. 2L Out-of-Plane[3], 2C Out-of-Plane[3]	$1\Delta_c$	$7\Delta_c$	0.5	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
3. HSS, Pipes, Tubes	$1\Delta_c$	$7\Delta_c$	0.5	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
c. Intermediate	Linear interpolation between the values for slender and stocky braces (after application of all applicable modifiers) shall be used.							
Braces in Tension (except EBF braces)[5]	$11\Delta_T$	$14\Delta_T$	0.8	$0.25\Delta_T$	$7\Delta_T$	$9\Delta_T$	$11\Delta_T$	$13\Delta_T$
Beams, Columns in Tension (except EBF beams, columns)[5]	$5\Delta_T$	$7\Delta_T$	1.0	$0.25\Delta_T$	$3\Delta_T$	$5\Delta_T$	$6\Delta_T$	$7\Delta_T$

[1]Δ_c is the axial deformation at expected buckling load.

[2]In addition to consideration of connection capacity in accordance with Section 5.5.2.4.1, values for braces shall be modified for connection robustness as follows: Where brace connections do not satisfy the requirements of AISC 341, Section 13.3c (AISC 2002), the acceptance criteria shall be multiplied by 0.8.

[3]Stitches for built-up members: Where the stitches for built-up braces do not satisfy the requirements of AISC 341, Section 13.2e (AISC 2002), the values of a, b, and all acceptance criteria shall be multiplied by 0.5.

[4]Section compactness: Modeling parameters and acceptance criteria apply to brace sections that are concrete-filled or seismically compact according to Table I-8-1 of AISC 341 (AISC 2002). Where the brace section is noncompact according to Table B5.1 of AISC *LRFD Specifications* (AISC 1999), the acceptance criteria shall be multiplied by 0.5. For intermediate compactness conditions, the acceptance criteria shall be multiplied by a value determined by linear interpolation between the seismically compact and the noncompact cases.

[5]Δ_T is the axial deformation at expected tensile yielding load.

[6]Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.

Practical Design to Prevent Progressive Collapse

- The following list should be considered for designing steel buildings with enhanced resistance to progressive collapse (Marchand and Alfawakhiri 2004):
  - **Beam design**
  - **Column design**
  - **Slab design**

1. Beam design

- Lateral support provided for full length of beam will prevent lateral-torsional-buckling. Loss of floor slab adjacent to a beam or change in support conditions can change the unbraced length and weaken the beam.
- Adding stiffener plates to specific beams will reduce local buckling.
- Using seismically compact sections is recommended.
- Beam should be laterally braced to reach plastic moment capacity in both positive and negative moments assuming that the slab is ineffective in lateral bracing.
- To prevent separation of beam from slab, use shear studs.
- Consider high-strength bolted connections to prevent brittle failure from concentrated stresses at weld locations. If welds are used, use notch-tough weld metal recommended for seismic design.
- Design connections for two limit states: 1) developing beam plastic moment and 2) developing beam axial tension capacity. Connections should be such that they permit large plastic deformations without brittle failures.
- Use moment connections for beams in both directions from perimeter, i.e., allow beams to cantilever from one bay in from the exterior.
- If possible make all beam-column connections fully restrained.
- For a composite floor system, design beams to be unshored rather than shored to provide extra strength in the beam.
- When using plastic analysis, ensure that local buckling or shear failure will not occur prior to developing full plastic moment capacity

2. Column design

- Check column stability for greater unbraced length due to loss of adjacent beams, increased axial load due to loss of adjacent columns, and for axial-moment interaction from beams delivering their plastic moment to the columns.
- Seismically compact columns may prevent local buckling under increased flexure
- If possible, use concrete-filled tube columns or concrete-encased shapes
- For built-up columns, use notch-tough weld metal.
- Columns should be designed stronger than beams to ensure plastic hinging of beams.
- Provide column stiffener plates (continuity plates) to prevent prying of column flanges when beams develop catenary tension. Stiffeners must be capable of transferring catenary tension from beam to beam across the column web.
- Add supplementary plates to column web.
- For narrow columns, provide lateral flange bracing to reduce unbraced length.
- Size column splices to develop axial tension capacity of columns and permit large plastic deformations. Use either bolted splices or welded splices with using notch-tough weld metal.

3. Slab design

- A concrete slab on metal deck can be used to provide full lateral support to beams.
- Use shear studs to maintain beam-to-deck connection.
- Provide additional reinforcing steel, bars in both directions as opposed to welded wire fabric to allow slab to develop adequate membrane capacity.
- Place reinforcement in slab center or use two layers of continuous bars.
- Lap reinforcement for continuity; do not use mechanical splices unless well staggered.
- Lightweight concrete floor slabs will reduce load but the blast resistance performance can be enhanced by use of normal weight concrete.
- Reinforce slab to carry self-weight in case of column or beam loss.

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