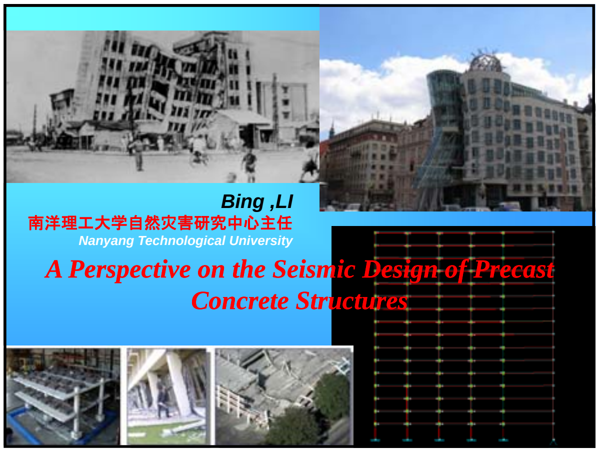
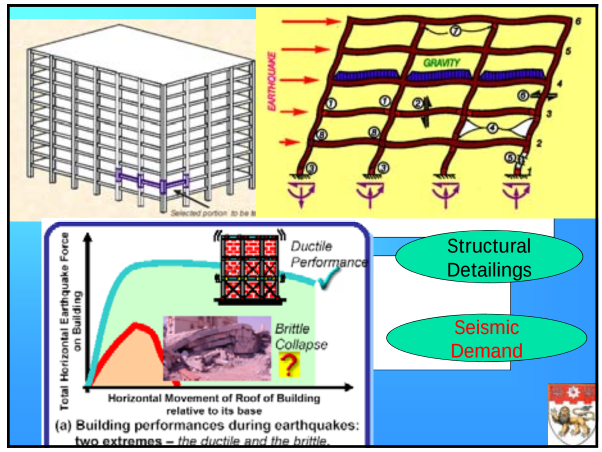
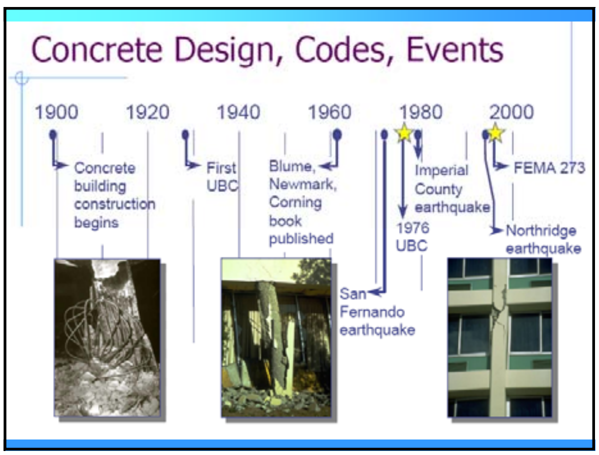
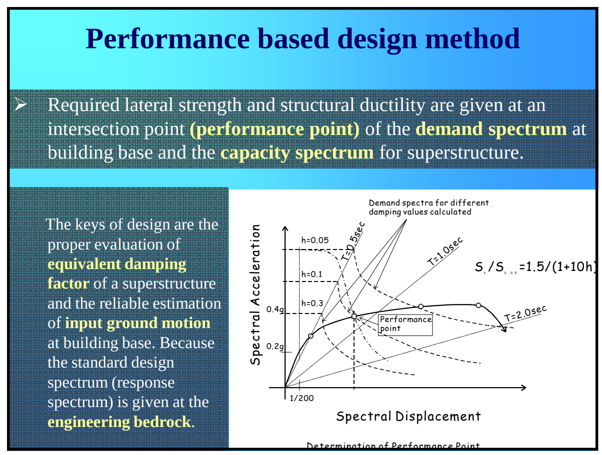
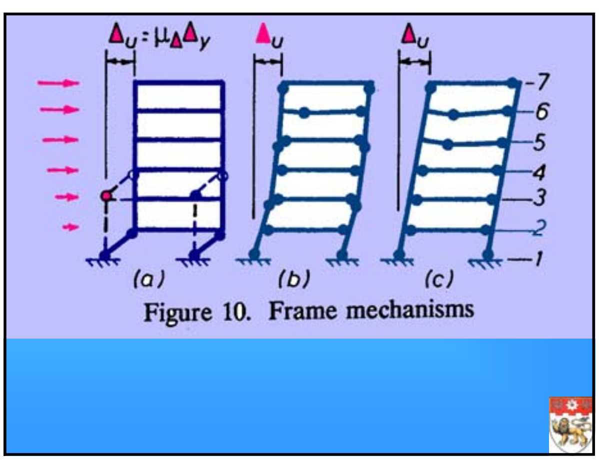

Bing ,LI
南洋理工大学自然灾害研究中心主任
Nanyang Technological University
A Perspective on the Seismic Design of Precast Concrete Structures


EARTHQUAKE
GRAVITY
Ductile Performance
Brittle Collapse
Total Horizontal Earthquake Force on Building
Horizontal Movement of Roof of Building relative to its base
(a) Building performances during earthquakes: two extremes – the ductile and the brittle
Structural Detailings
Seismic Demand

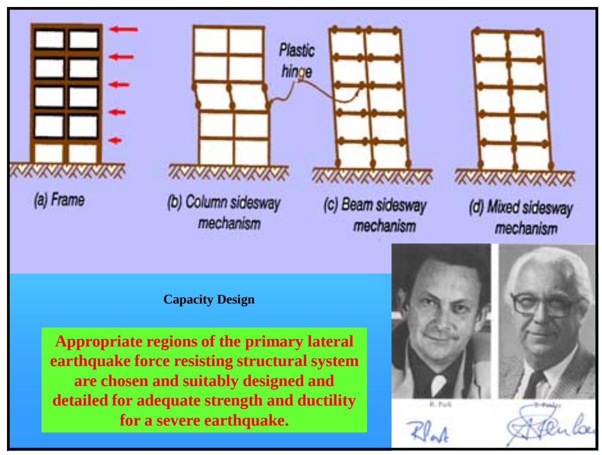

Concrete Design, Codes, Events
1900
1920
1940
1960
1980
2000
Concrete building construction begins
First UBC
Blume, Newmark, Corning book published
Imperial County earthquake
FEMA 273
1976 UBC
Northridge earthquake
San Fernando earthquake

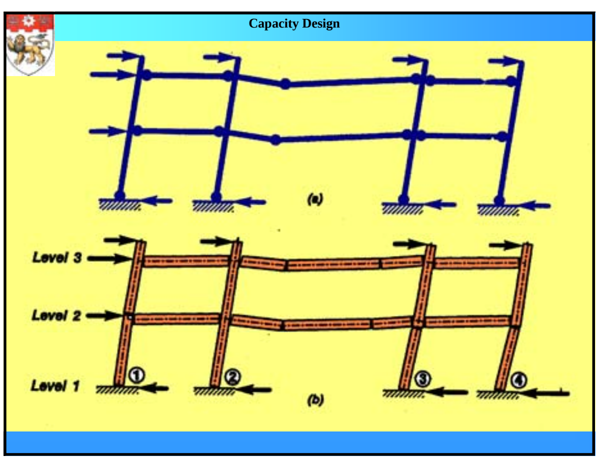

Performance based design method
Required lateral strength and structural ductility are given at an intersection point (performance point) of the demand spectrum at building base and the capacity spectrum for superstructure.
The keys of design are the proper evaluation of equivalent damping factor of a superstructure and the reliable estimation of input ground motion at building base. Because the standard design spectrum (response spectrum) is given at the engineering bedrock.
Demand spectra for different damping values calculated
Spectral Acceleration
Spectral Displacement
Performance point
T=1.0sec
T=2.0sec
h=0.05
h=0.1
h=0.3
1/200

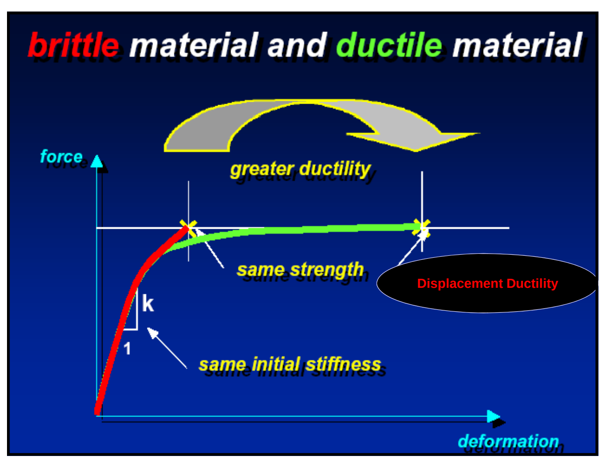

(a)
(b)
(c)
Figure 10. Frame mechanisms

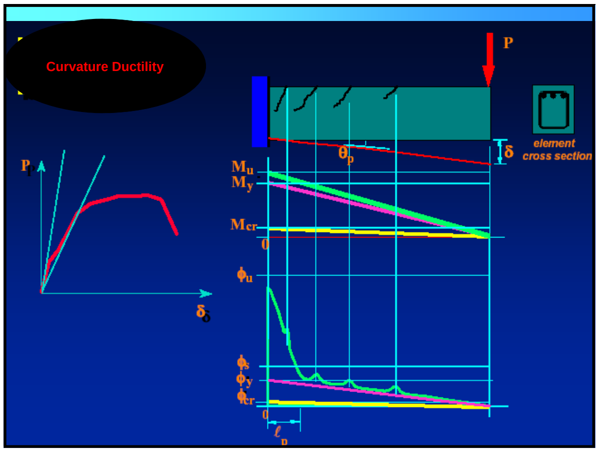

Poor Performance

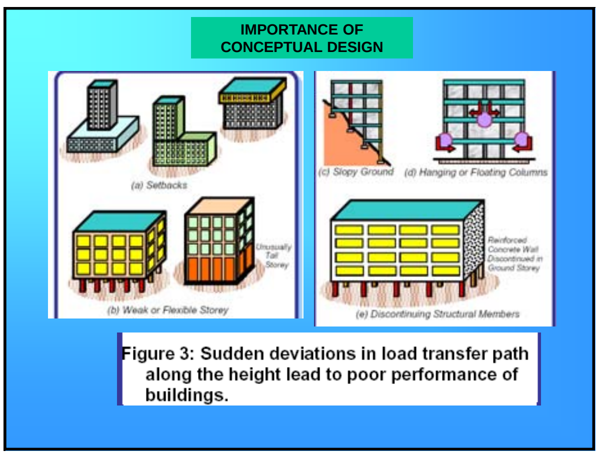

Poor Performance

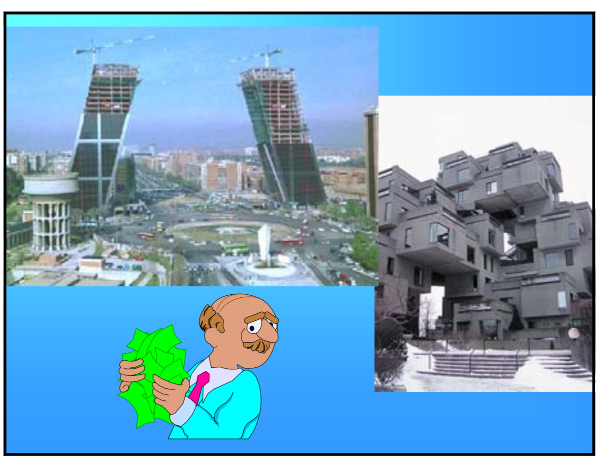

Plastic hinge
(a) Frame
(b) Column sidesway mechanism
(c) Beam sidesway mechanism
(d) Mixed sidesway mechanism
Capacity Design
Appropriate regions of the primary lateral earthquake force resisting structural system are chosen and suitably designed and detailed for adequate strength and ductility for a severe earthquake.

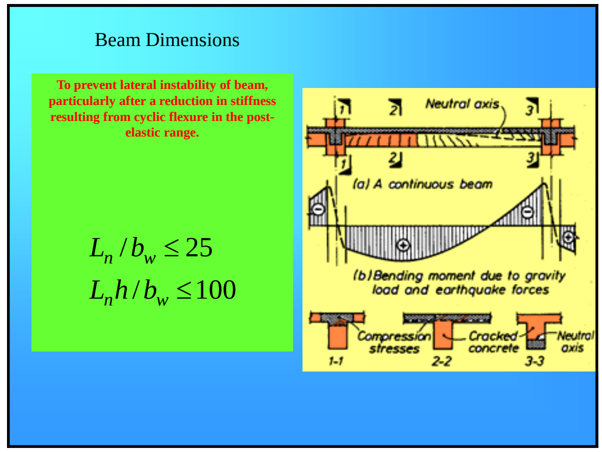

Capacity Design
(a)
Level 3
Level 2
Level 1
(b)

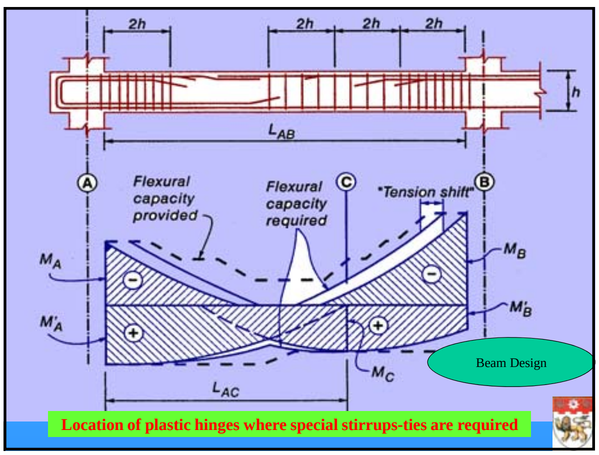

brittle material and ductile material
force
greater ductility
same strength
Displacement Ductility
k
1
same initial stiffness
deformation

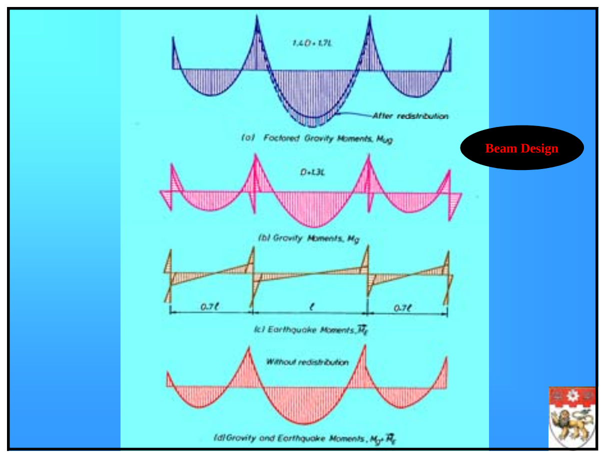

Curvature Ductility
P
element
cross section
δ
Mu
My
Mcr

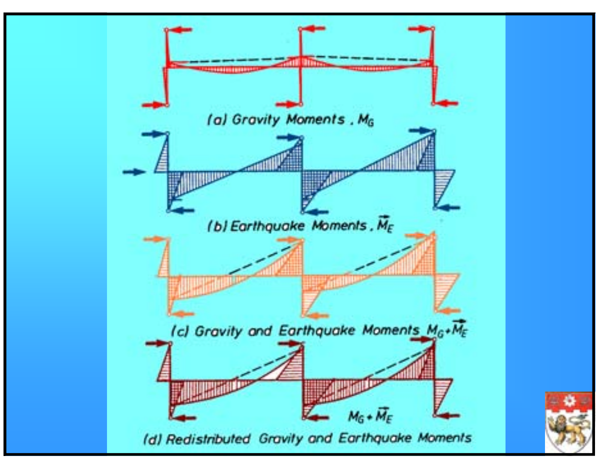

IMPORTANCE OF CONCEPTUAL DESIGN
(a) Setbacks
(b) Weak or Flexible Storey
Unusually Tall Storey
(c) Slopy Ground
(d) Hanging or Floating Columns
Reinforced Concrete Wall Discontinued in Ground Storey
(e) Discontinuing Structural Members
Figure 3: Sudden deviations in load transfer path along the height lead to poor performance of buildings.

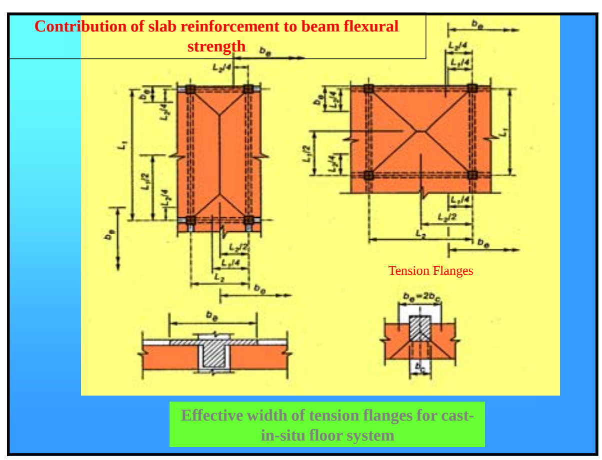



Beam Dimensions
To prevent lateral instability of beam, particularly after a reduction in stiffness resulting from cyclic flexure in the post-elastic range.
$L_n / b_w \le 25$
$L_n h / b_w \le 100$
Neutral axis
(a) A continuous beam
(b) Bending moment due to gravity load and earthquake forces
Compression stresses
Cracked concrete
Neutral axis
1-1
2-2
3-3

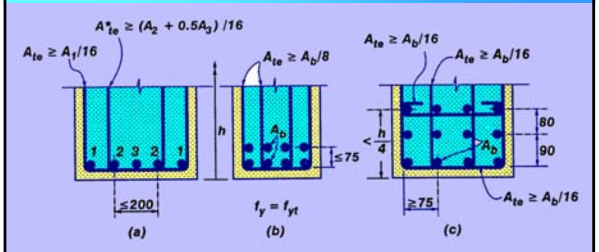

2h
2h
2h
2h
h
L_{AB}
Flexural capacity provided
Flexural capacity required
"Tension shift"
M_A
M'_A
M_B
M'_B
M_C
L_{AC}
Beam Design
Location of plastic hinges where special stirrups-ties are required


After redistribution
(a) Factored Gravity Moments, M_{ug}
Beam Design
(b) Gravity Moments, M_g
0.7ℓ
ℓ
0.7ℓ
(c) Earthquake Moments
Without redistribution


(a) Gravity Moments, M_G
(b) Earthquake Moments, $\vec{M}_E$
(c) Gravity and Earthquake Moments $M_G + \vec{M}_E$
$M_G + \vec{M}_E$
(d) Redistributed Gravity and Earthquake Moments


Contribution of slab reinforcement to beam flexural strength
Tension Flanges
Effective width of tension flanges for cast-in-situ floor system

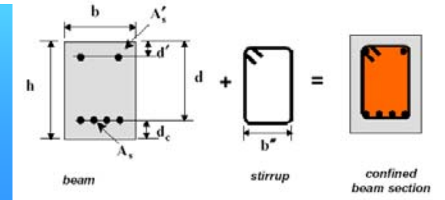
Transverse Reinforcement in Beams



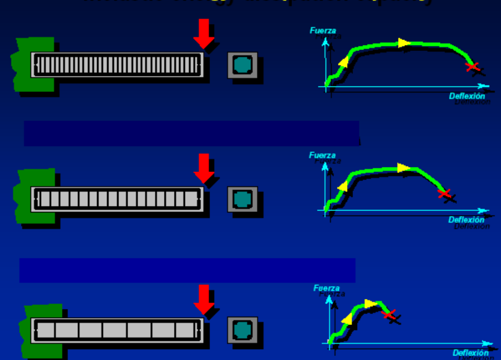
Example of stirrup-ties to prevent premature buckling of longitudinal bars of beams in potential plastic hinge zones of beams

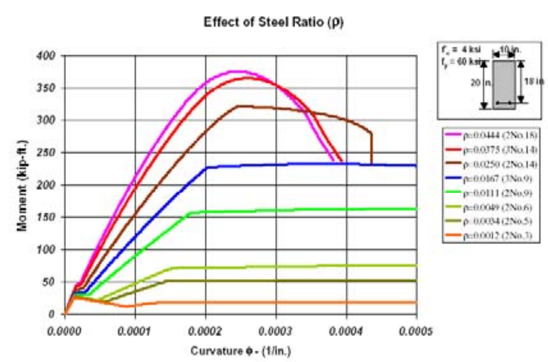
Effect of Confinement Reinforcement In Beam Behavior

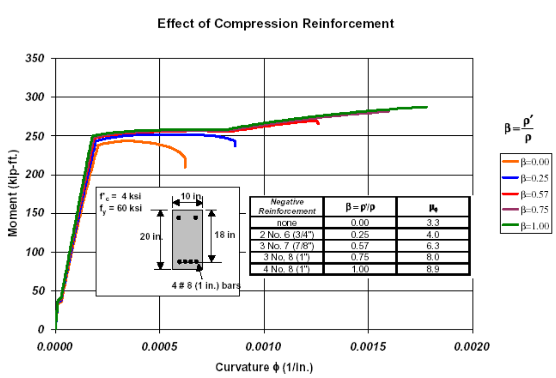
If the compression zone of a member is confined by closely spaced transverse reinforcement in the form of closed stirrups, ties, hoops or spirals, the ductility of the concrete may be greatly improved and a more ductile performance of the member at the ultimate load will result.



Inelastic energy dissipation capacity







Negative Reinforcement	$\beta = \rho'/\rho$	μ_ϕ
none	0.00	3.3
2 No. 6 (3/4")	0.25	4.0
3 No. 7 (7/8")	0.57	6.3
3 No. 8 (1")	0.75	8.0
4 No. 8 (1")	1.00	8.9

The Control of Shear Strength

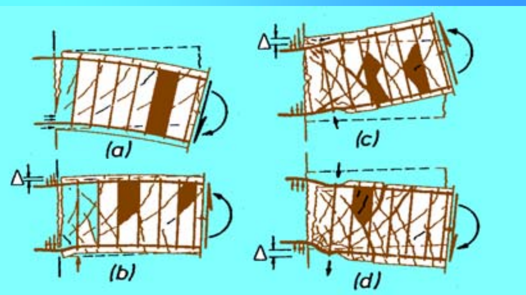
Because of the low and unreliable tensile strength of concrete, shear strength in seismic design must be based on mechanisms that can be mobilized after the onset of cracking. Such mechanisms, relying on the contribution of concrete in diagonal compression only, are well established.

$$A_v = \frac{V^* s}{d f_{yt}}$$

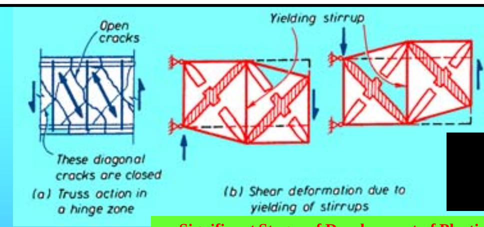
It should be the designer's intention to retain essentially elastic response in shear

$$V_u^* = \frac{M'_{oA} + M_{oB}}{L_{AB}} + \frac{wL_{AB}}{2}$$

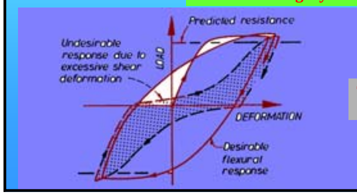
Failure by sliding shear is a feature of inelastic seismic response.



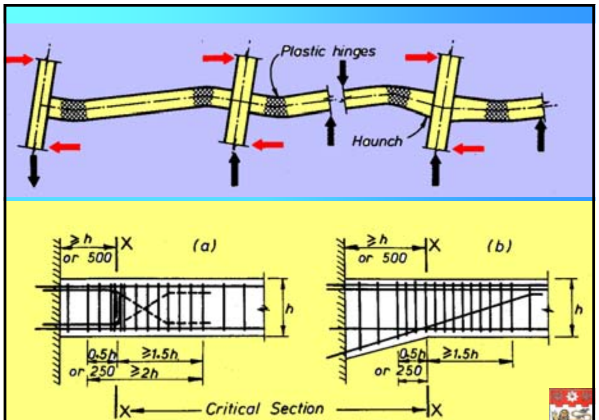
Inelastic flexural response may lead to sliding shear failure



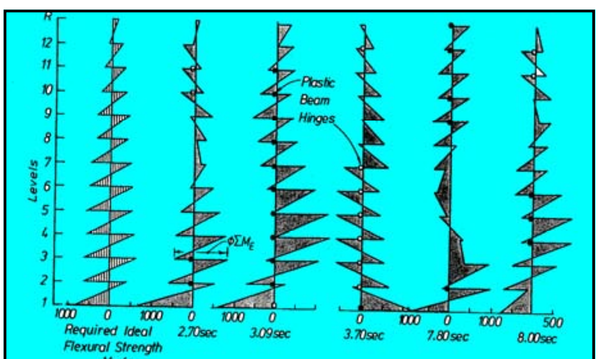
Significant Stages of Development of Plastic Hinge During Cyclic Flexure with High Shear



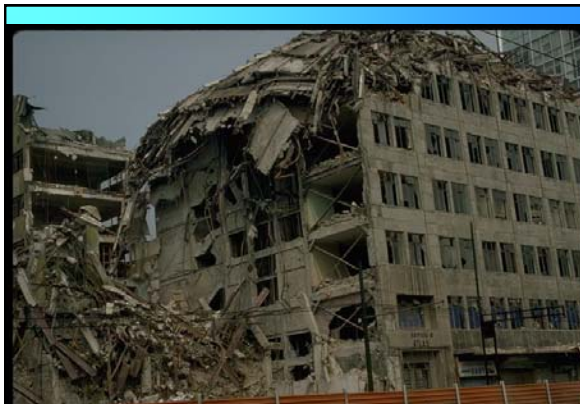
The effect on hysteretic response of inelastic shear deformations

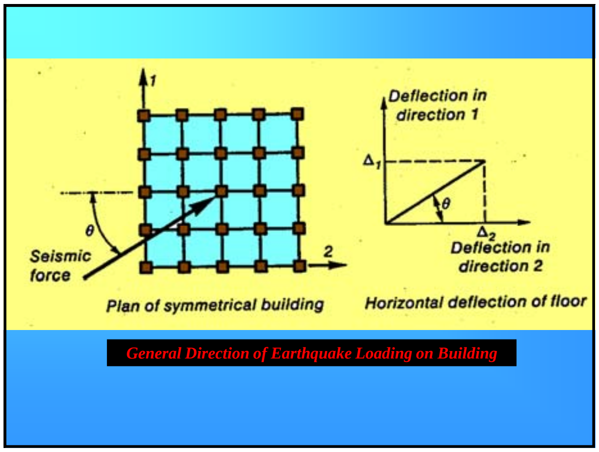


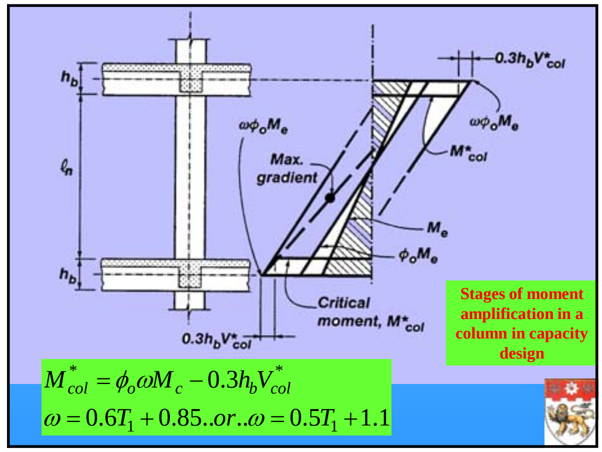
Principles of Column Design

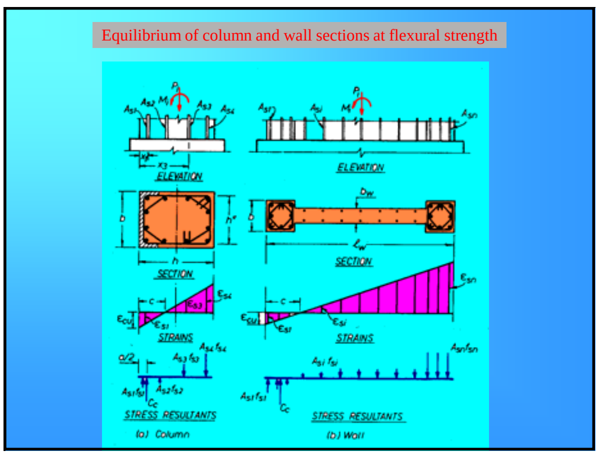


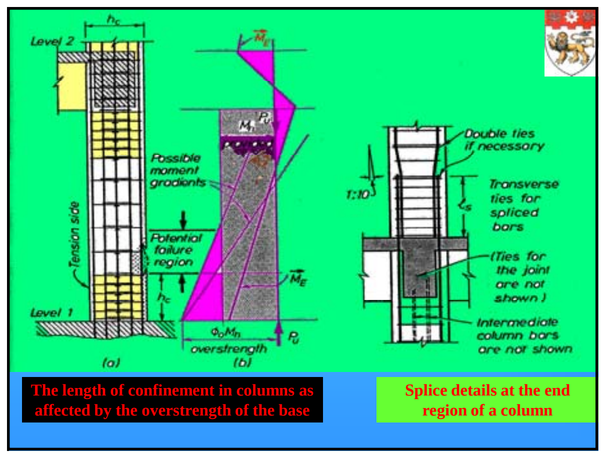
A comparison of column patterns due to horizontal static and dynamic forces

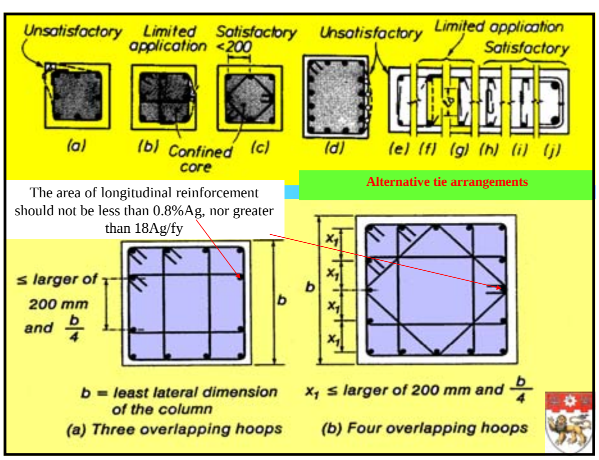


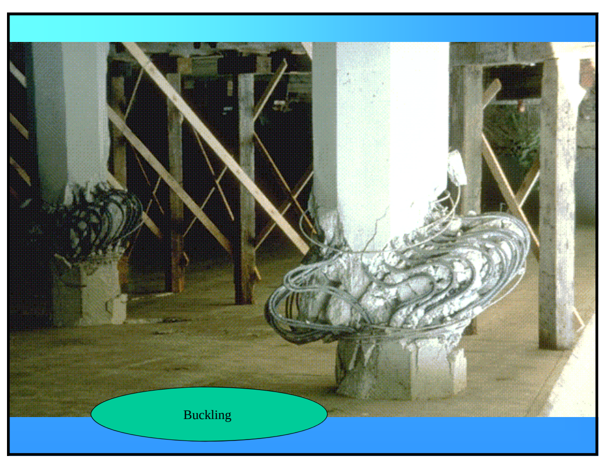

Deflection in direction 1
Seismic force
Deflection in direction 2
Plan of symmetrical building
Horizontal deflection of floor
General Direction of Earthquake Loading on Building

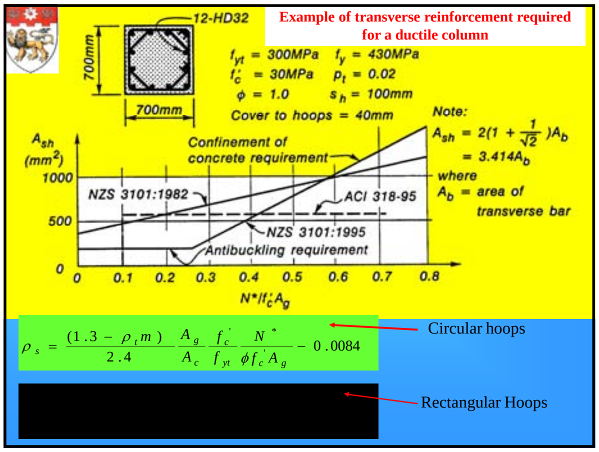

Max. gradient
Critical moment, M*col
Stages of moment amplification in a column in capacity design
$M^*_{col} = \phi_o \omega M_c - 0.3 h_b V^*_{col}$
$\omega = 0.6T_1 + 0.85..or..\omega = 0.5T_1 + 1.1$


Equilibrium of column and wall sections at flexural strength
ELEVATION
SECTION
STRAINS
STRESS RESULTANTS
(a) Column
(b) Wall

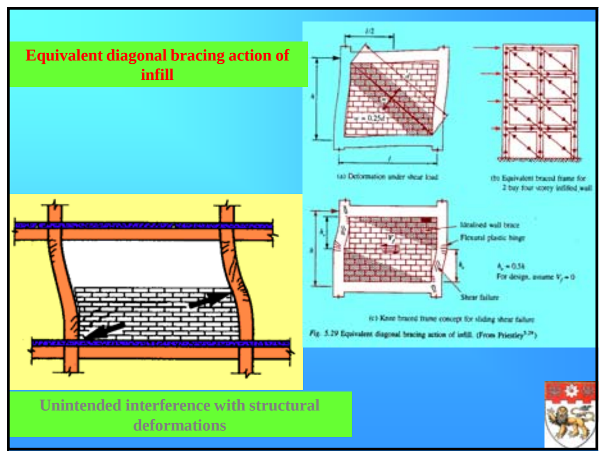

Level 2
Level 1
Tension side
Possible moment gradients
Potential failure region
overstrength
Double ties if necessary
Transverse ties for spliced bars
(Ties for the joint are not shown)
Intermediate column bars are not shown
The length of confinement in columns as affected by the overstrength of the base
Splice details at the end region of a column

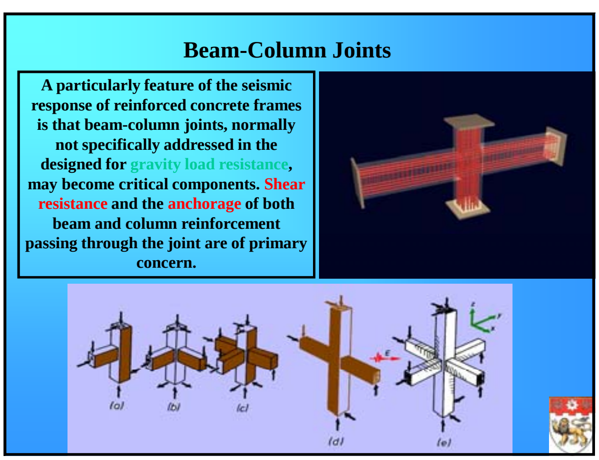

Unsatisfactory
Limited application
Satisfactory <200
Unsatisfactory
Limited application
Satisfactory
Confined core
Alternative tie arrangements
The area of longitudinal reinforcement should not be less than 0.8%Ag, nor greater than 18Ag/fy
≤ larger of 200 mm and b/4
b = least lateral dimension of the column
x1 ≤ larger of 200 mm and b/4
(a) Three overlapping hoops
(b) Four overlapping hoops

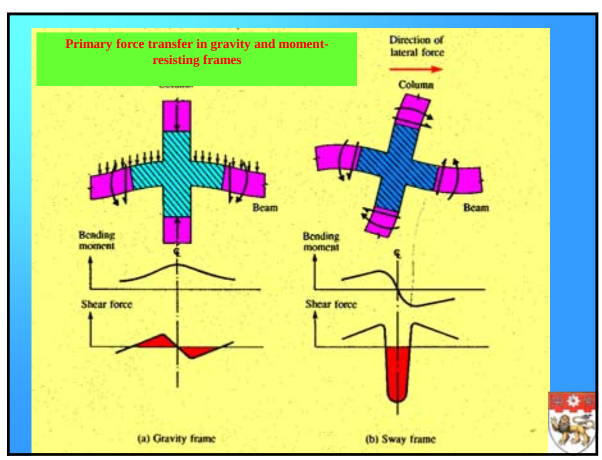

Buckling

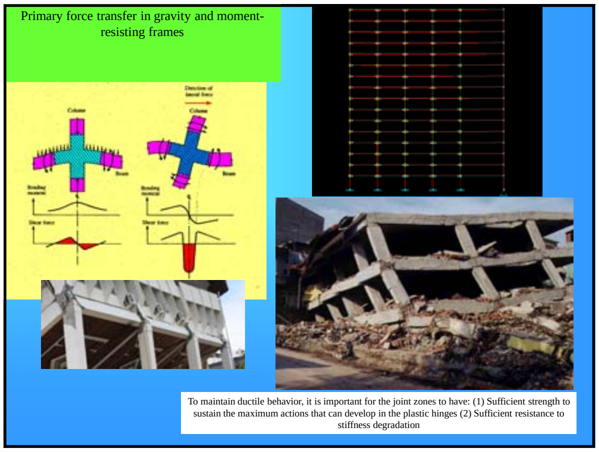

Example of transverse reinforcement required for a ductile column
12-HD32
700mm
700mm
f_{yt} = 300MPa f_y = 430MPa
f'_c = 30MPa p_t = 0.02
ϕ = 1.0 s_h = 100mm
Cover to hoops = 40mm
Note:
$A_{sh} = 2(1 + \frac{1}{\sqrt{2}})A_b = 3.414A_b$
where A_b = area of transverse bar
A_{sh} (mm^2)
Confinement of concrete requirement
NZS 3101:1982
ACI 318-95
NZS 3101:1995
Antibuckling requirement
N^*/f'_cA_g
$\rho_s = \frac{(1.3 - \rho_t m)}{2.4} \frac{A_g}{A_c} \frac{f'_c}{f_{yt}} \frac{N^*}{\phi f'_c A_g} - 0.0084$
Circular hoops
[redacted]
Rectangular Hoops

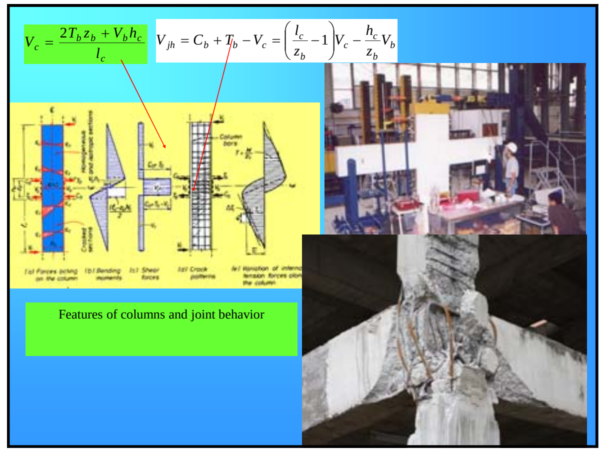

Short Columns

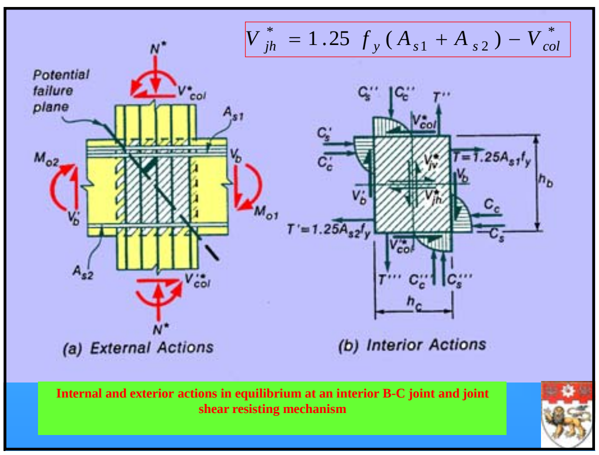

Equivalent diagonal bracing action of infill
Unintended interference with structural deformations

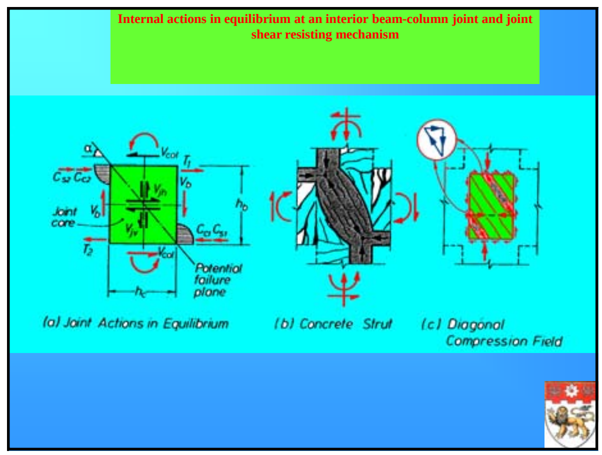

Beam-Column Joints
A particularly feature of the seismic response of reinforced concrete frames is that beam-column joints, normally not specifically addressed in the designed for gravity load resistance, may become critical components. Shear resistance and the anchorage of both beam and column reinforcement passing through the joint are of primary concern.
(a)
(b)
(c)
(d)
(e)

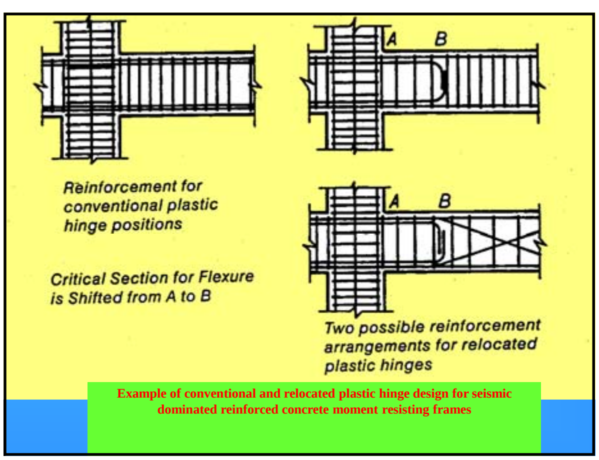

Primary force transfer in gravity and moment-resisting frames
Direction of lateral force
Column
Beam
Bending moment
Shear force
(a) Gravity frame
(b) Sway frame

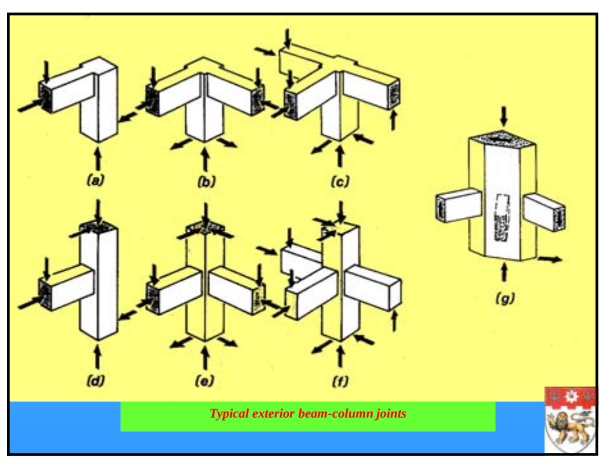

Primary force transfer in gravity and moment-resisting frames
To maintain ductile behavior, it is important for the joint zones to have: (1) Sufficient strength to sustain the maximum actions that can develop in the plastic hinges (2) Sufficient resistance to stiffness degradation

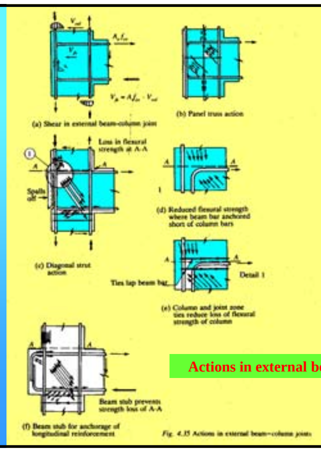

$V_c = \frac{2T_b z_b + V_b h_c}{l_c}$
$V_{jh} = C_b + T_b - V_c = \left(\frac{l_c}{z_b} - 1\right)V_c - \frac{h_c}{z_b}V_b$
Features of columns and joint behavior


$V^*_{jh} = 1.25 f_y (A_{s1} + A_{s2}) - V^*_{col}$
Potential failure plane
(a) External Actions
(b) Interior Actions
Internal and exterior actions in equilibrium at an interior B-C joint and joint shear resisting mechanism

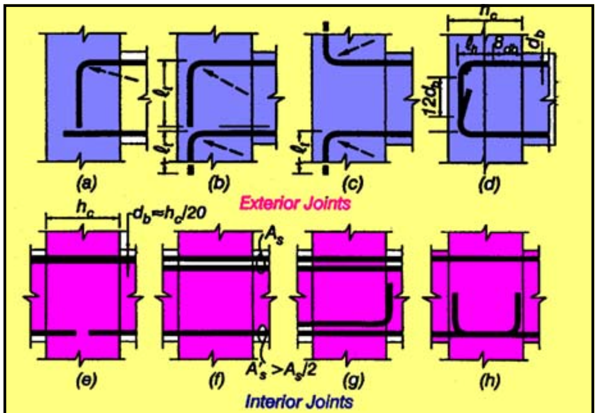

Internal actions in equilibrium at an interior beam-column joint and joint shear resisting mechanism
Joint core
Potential failure plane
(a) Joint Actions in Equilibrium
(b) Concrete Strut
(c) Diagonal Compression Field


Reinforcement for conventional plastic hinge positions
Critical Section for Flexure is Shifted from A to B
Two possible reinforcement arrangements for relocated plastic hinges
Example of conventional and relocated plastic hinge design for seismic dominated reinforced concrete moment resisting frames

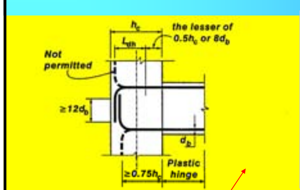

(a)
(b)
(c)
(d)
(e)
(f)
(g)
Typical exterior beam-column joints

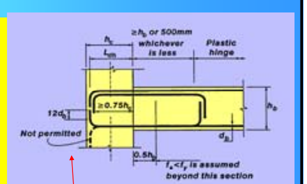


Actions in external beam-column joints



Anchorage of beam bars in beam-column joints



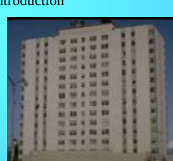


Anchorage of beam bars when the critical section of the plastic hinge forms at the column face

Anchorage of beam bars when the critical section of the plastic hinge is located at sufficient distance from the column face

Research on Assessment of Existing RC Structural Walls

Introduction

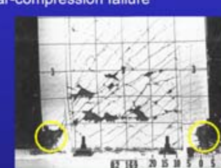


Structural walls can provide structures under service loading with sufficient stiffness, minimizing deformations and damage to non-structural elements. Under severe seismic excitations, they can provide sufficient strength, energy absorption and dissipation capacities to prevent collapse and loss of life.

Shear Modes of Failure

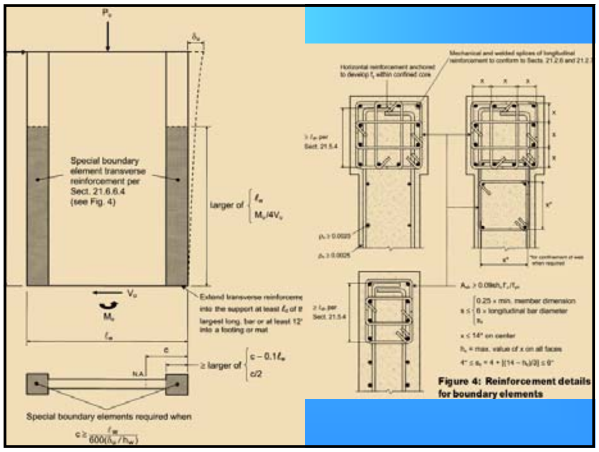
- Shear-tension failure

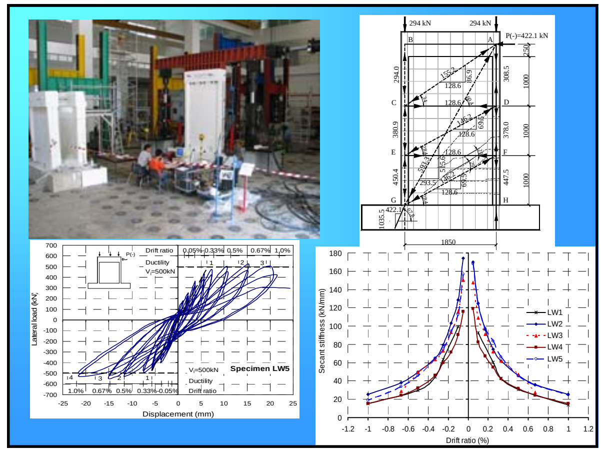
- Shear-compression failure

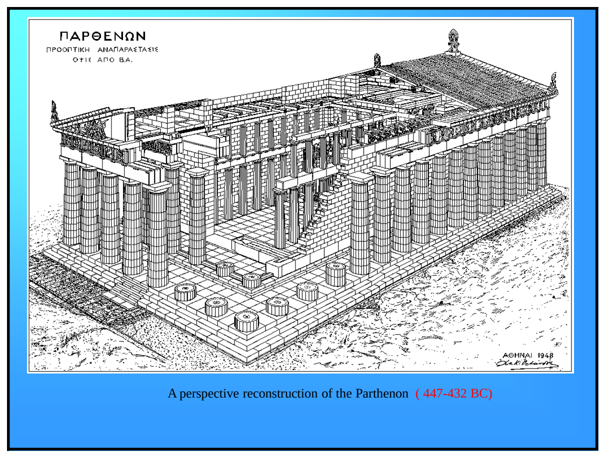


- Diagonal-compression failure (web crushing)

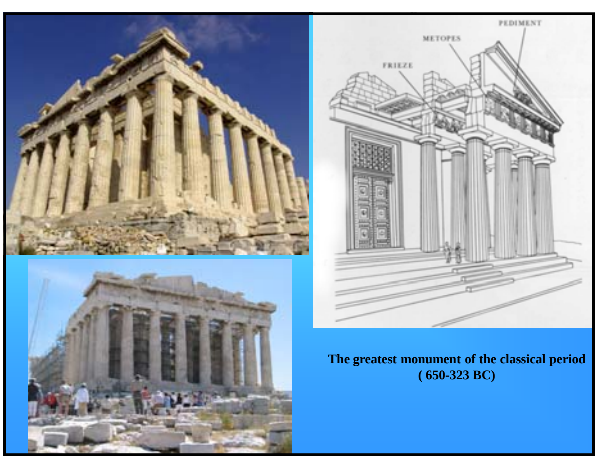
Sliding-shear failure



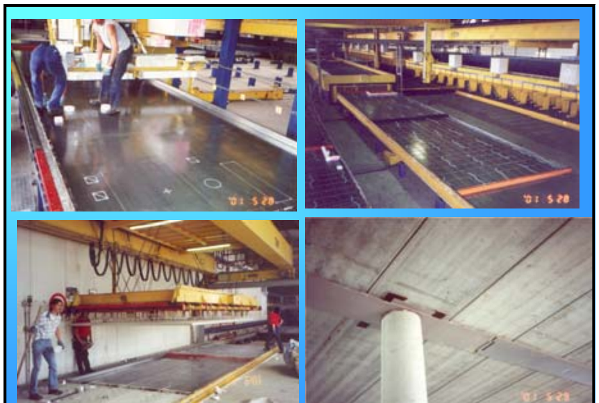




A perspective reconstruction of the Parthenon (447-432 BC)



The greatest monument of the classical period (650-323 BC)



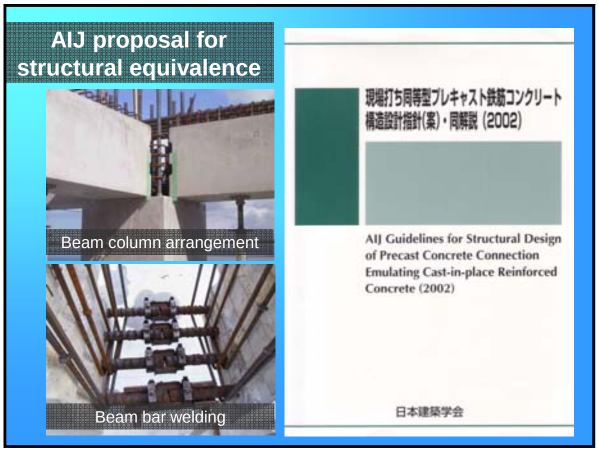
PROBLEMS WITH PRECAST FRAMES

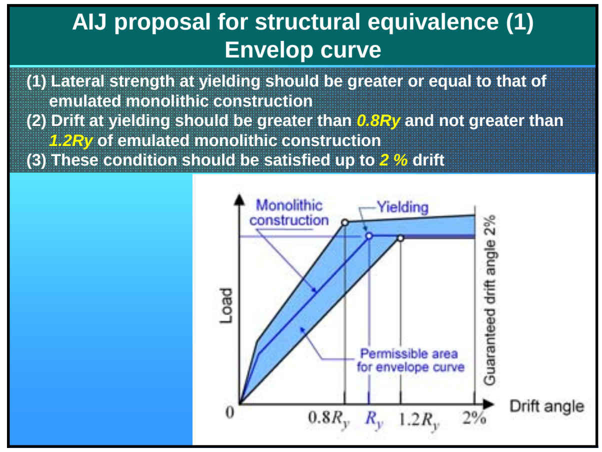
Total Collapse of Precast Industrial Buildings

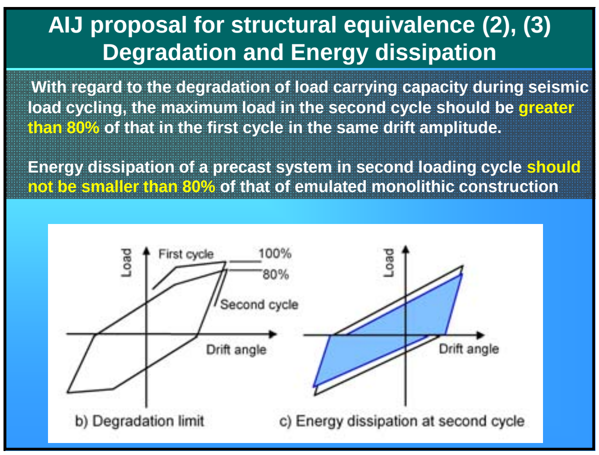
Precast concrete construction

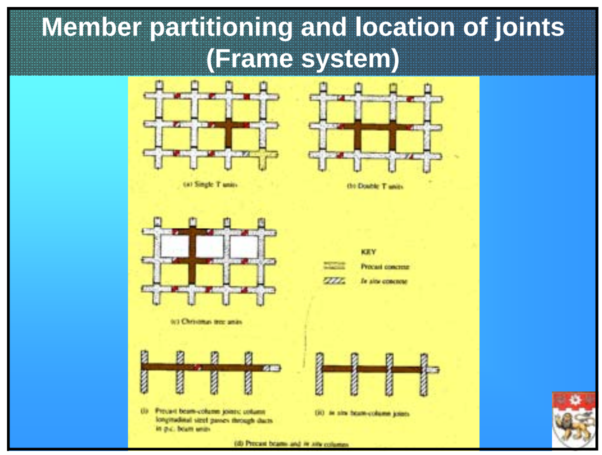
Precast reinforced concrete buildings are designed and constructed that attempt *to emulate seismic performance of cast-in-place monolithic structures.*

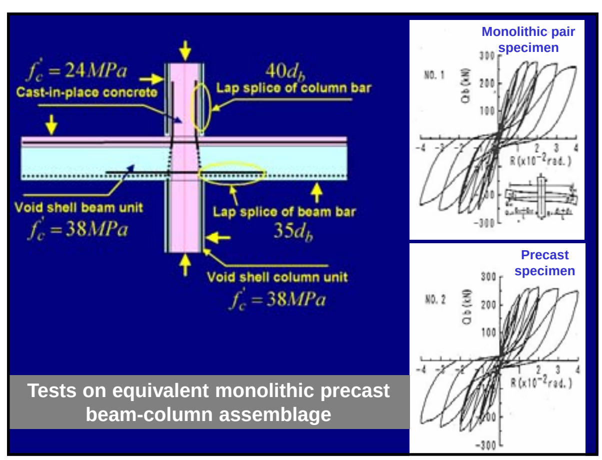
- Equivalent monolithic structural behaviour is generally demonstrated by *tests on precast beam-column sub-assemblages.*
- Experimentally observed data is compared with that of simultaneously constructed pair specimen or with past experimental data in view of *lateral stiffness, lateral strength, structural ductility and hysteretic behaviour (energy dissipation).*

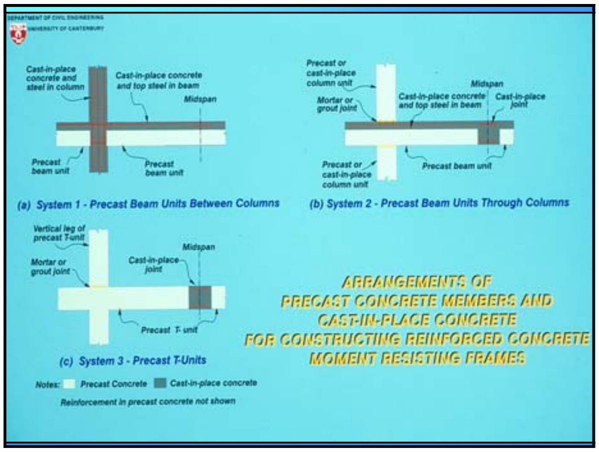

AIJ proposal for structural equivalence
Beam column arrangement
Beam bar welding
現場打ち同等型プレキャスト鉄筋コンクリート構造設計指針(案)・同解説 (2002)
AIJ Guidelines for Structural Design of Precast Concrete Connection Emulating Cast-in-place Reinforced Concrete (2002)
日本建築学会

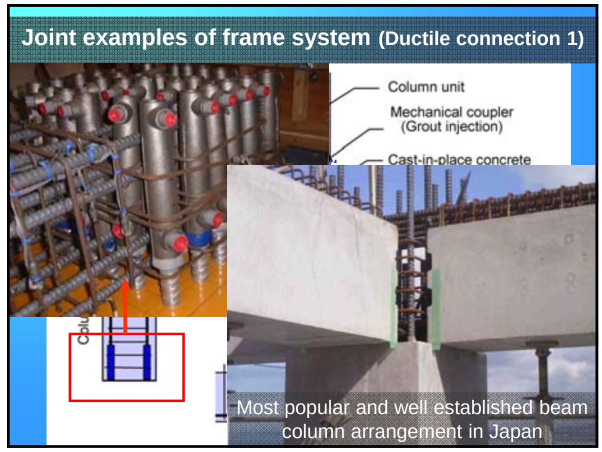

AIJ proposal for structural equivalence (1)
Envelop curve
(1) Lateral strength at yielding should be greater or equal to that of emulated monolithic construction
(2) Drift at yielding should be greater than 0.8Ry and not greater than 1.2Ry of emulated monolithic construction
(3) These condition should be satisfied up to 2 % drift
Monolithic construction
Yielding
Load
Guaranteed drift angle 2%
Permissible area for envelope curve
0
0.8Ry
Ry
1.2Ry
2%
Drift angle

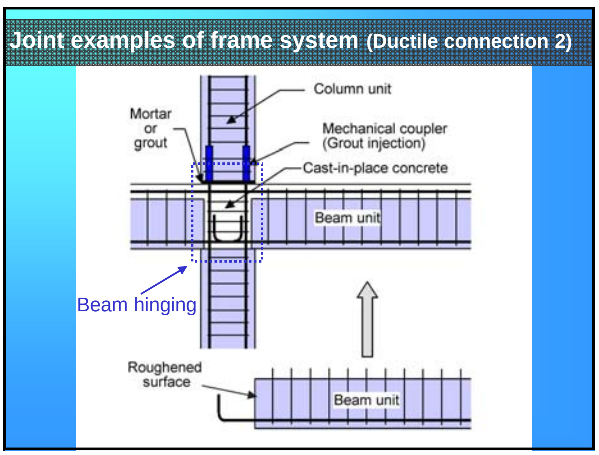

AIJ proposal for structural equivalence (2), (3)
Degradation and Energy dissipation
With regard to the degradation of load carrying capacity during seismic load cycling, the maximum load in the second cycle should be greater than 80% of that in the first cycle in the same drift amplitude.
Energy dissipation of a precast system in second loading cycle should not be smaller than 80% of that of emulated monolithic construction
Load
First cycle
100%
80%
Second cycle
Drift angle
Load
Drift angle
b) Degradation limit
c) Energy dissipation at second cycle

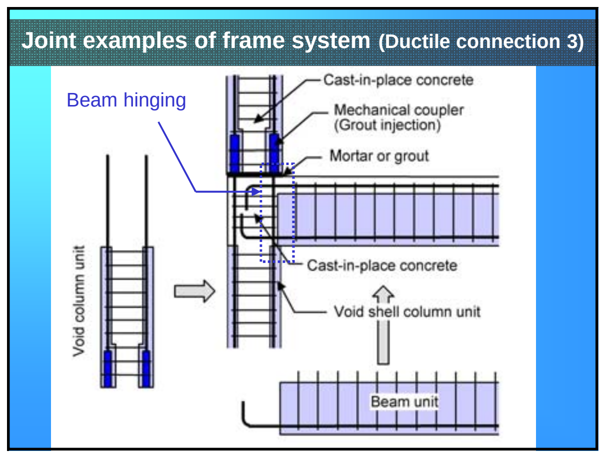

Member partitioning and location of joints
(Frame system)

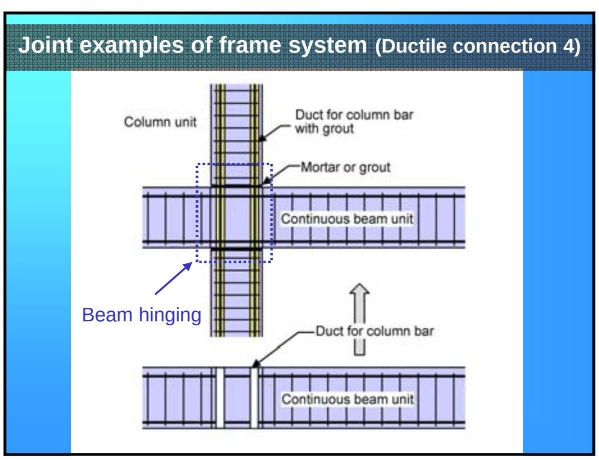

f'c = 24MPa
Cast-in-place concrete
40db
Lap splice of column bar
Void shell beam unit
f'c = 38MPa
Lap splice of beam bar
35db
Void shell column unit
f'c = 38MPa
Monolithic pair specimen
Precast specimen
Tests on equivalent monolithic precast beam-column assemblage

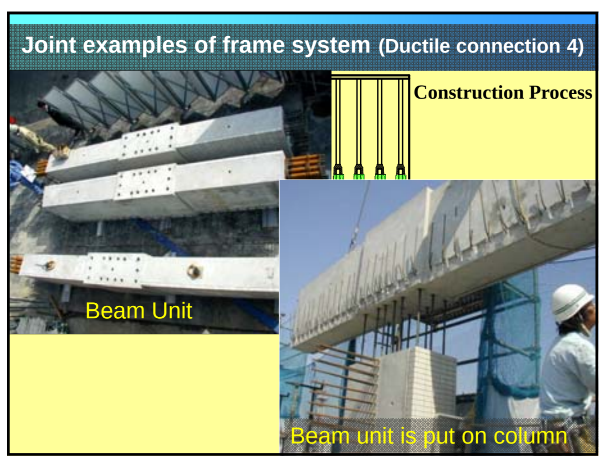

DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF CANTERBURY
Cast-in-place concrete and steel in column
Cast-in-place concrete and top steel in beam
Midspan
Precast beam unit
Precast beam unit
(a) System 1 - Precast Beam Units Between Columns
Precast or cast-in-place column unit
Mortar or grout joint
Cast-in-place concrete and top steel in beam
Midspan
Cast-in-place joint
Precast or cast-in-place column unit
Precast beam unit
(b) System 2 - Precast Beam Units Through Columns
Vertical leg of precast T-unit
Mortar or grout joint
Cast-in-place joint
Midspan
Precast T- unit
(c) System 3 - Precast T-Units
Notes: Precast Concrete Cast-in-place concrete
Reinforcement in precast concrete not shown
ARRANGEMENTS OF PRECAST CONCRETE MEMBERS AND CAST-IN-PLACE CONCRETE FOR CONSTRUCTING REINFORCED CONCRETE MOMENT RESISTING FRAMES

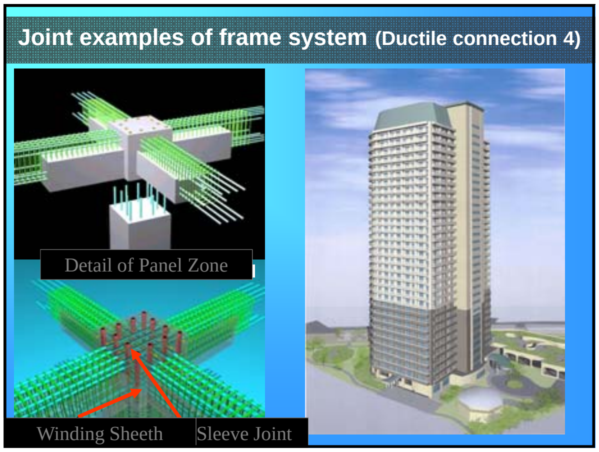

Joint examples of frame system (Ductile connection 1)
Column unit
Mechanical coupler (Grout injection)
Cast-in-place concrete
Most popular and well established beam column arrangement in Japan


Joint examples of frame system (Ductile connection 2)
Column unit
Mortar or grout
Mechanical coupler (Grout injection)
Cast-in-place concrete
Beam unit
Beam hinging
Roughened surface
Beam unit

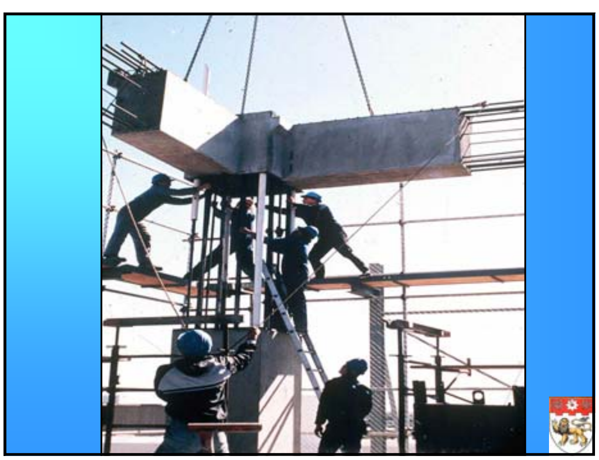

Joint examples of frame system (Ductile connection 3)
Beam hinging
Cast-in-place concrete
Mechanical coupler (Grout injection)
Mortar or grout
Void column unit
Cast-in-place concrete
Void shell column unit
Beam unit

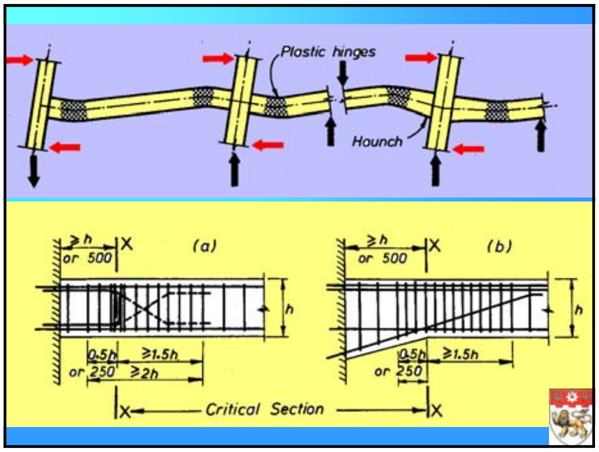

Joint examples of frame system (Ductile connection 4)
Column unit
Duct for column bar with grout
Mortar or grout
Continuous beam unit
Beam hinging
Duct for column bar
Continuous beam unit


Joint examples of frame system (Ductile connection 4)
Construction Process
Beam Unit
Beam unit is put on column

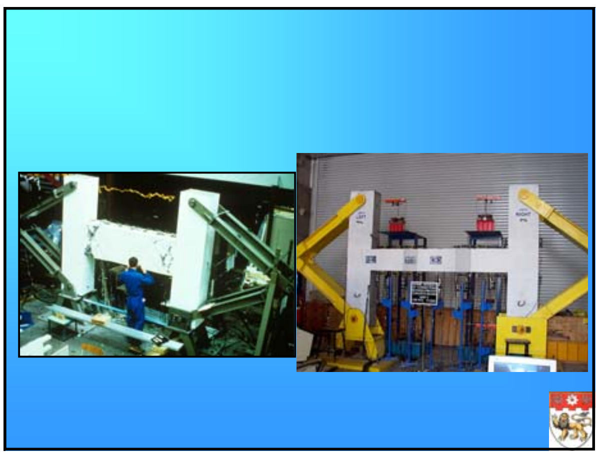

Joint examples of frame system (Ductile connection 4)
Detail of Panel Zone
Winding Sheeth
Sleeve Joint

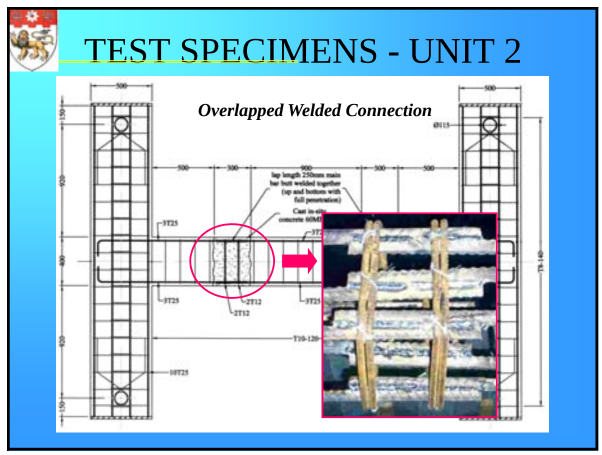

Innovative Construction Staging Operations

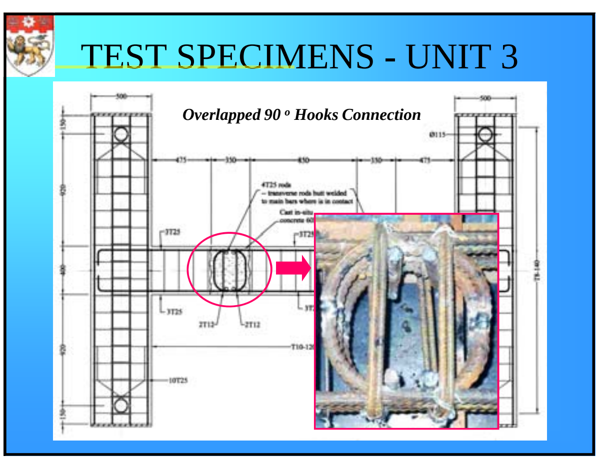


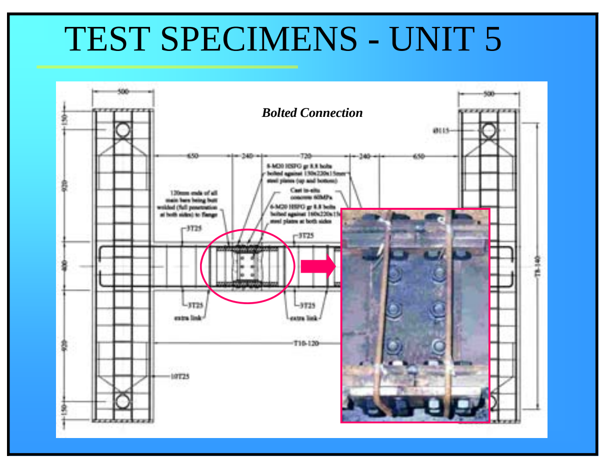

Plastic hinges
Haunch
≥h or 500
(a)
(b)
0.5h or 250
≥1.5h ≥2h
Critical Section

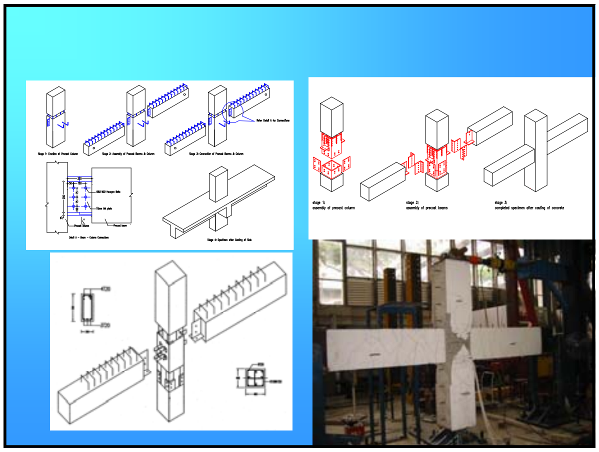

Joint examples of frame system (Strong connection)
Strong connection with shear keys and re-bar welding at beam center

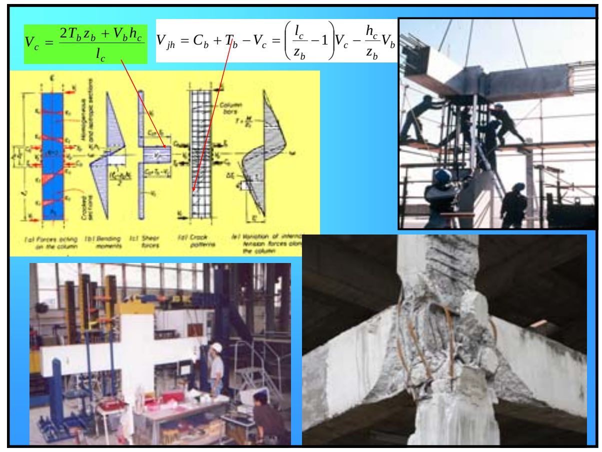


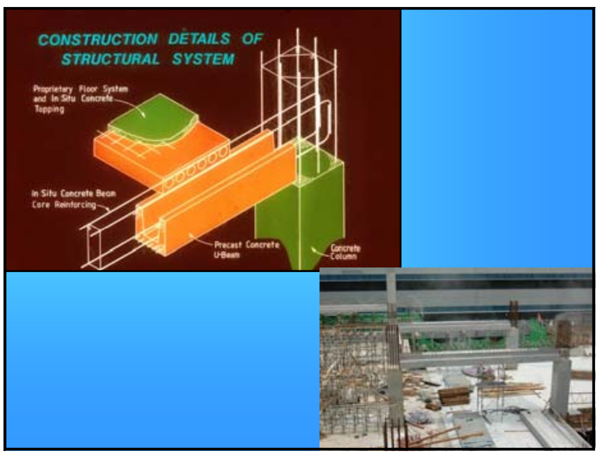

TEST SPECIMENS - UNIT 2
Overlapped Welded Connection

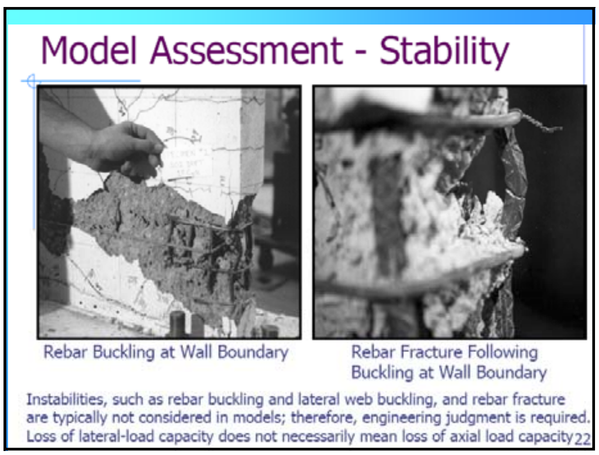

TEST SPECIMENS - UNIT 3
Overlapped 90 º Hooks Connection

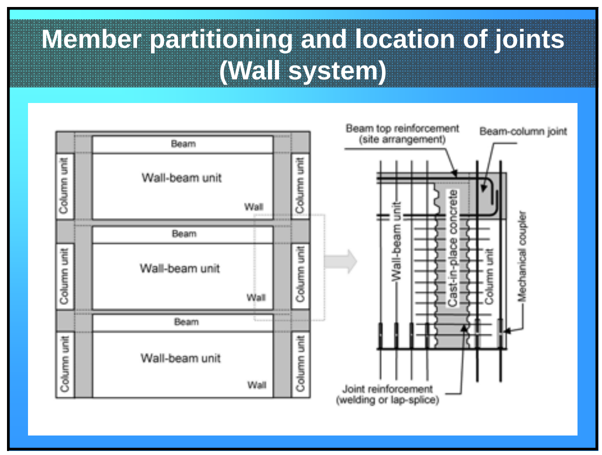

TEST SPECIMENS - UNIT 5
Bolted Connection

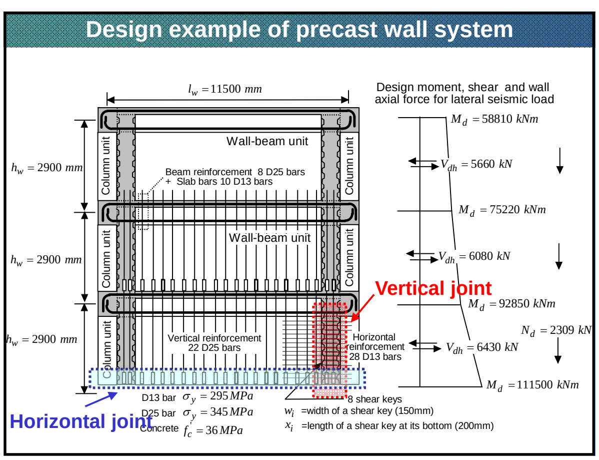


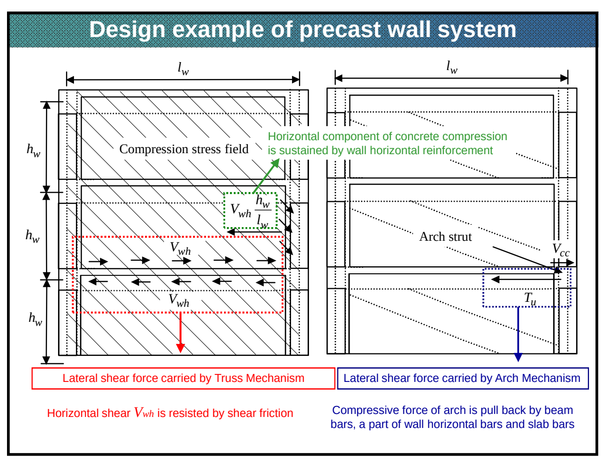

$V_c = \frac{2T_b z_b + V_b h_c}{l_c}$
$V_{jh} = C_b + T_b - V_c = \left(\frac{l_c}{z_b} - 1\right)V_c - \frac{h_c}{z_b}V_b$


CONSTRUCTION DETAILS OF
STRUCTURAL SYSTEM
Proprietary Floor System and In-Situ Concrete Topping
In Situ Concrete Beam Core Reinforcing
Precast Concrete U-Beam
Concrete Column

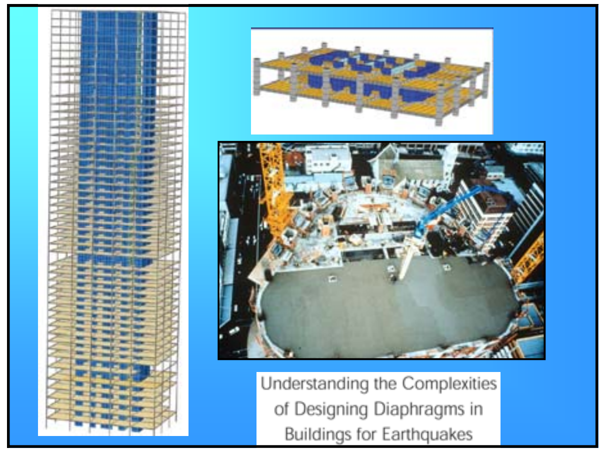

Model Assessment - Stability
Rebar Buckling at Wall Boundary
Rebar Fracture Following Buckling at Wall Boundary
Instabilities, such as rebar buckling and lateral web buckling, and rebar fracture are typically not considered in models; therefore, engineering judgment is required.
Loss of lateral-load capacity does not necessarily mean loss of axial load capacity
22

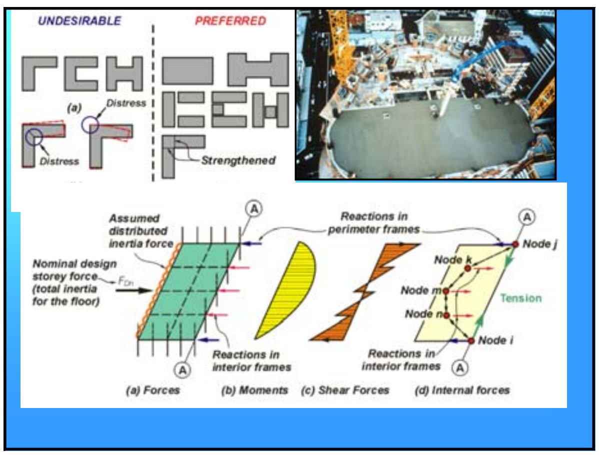

Member partitioning and location of joints (Wall system)
Beam
Wall-beam unit
Wall
Column unit
Beam top reinforcement (site arrangement)
Beam-column joint
Wall-beam unit
Cast-in-place concrete
Column unit
Mechanical coupler
Joint reinforcement (welding or lap-splice)

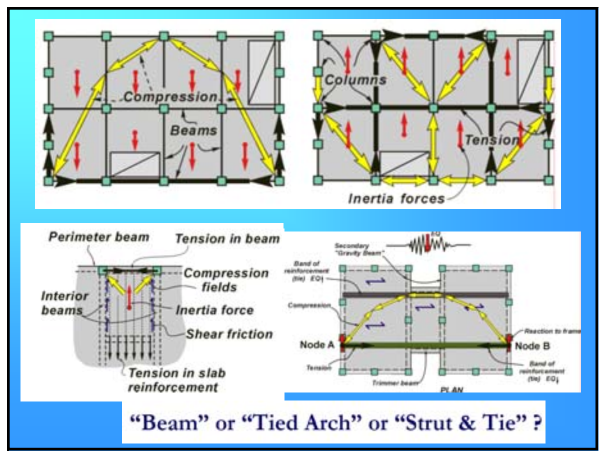

Design example of precast wall system
$l_w = 11500\ mm$
Design moment, shear and wall axial force for lateral seismic load
$h_w = 2900\ mm$
Wall-beam unit
Column unit
Beam reinforcement 8 D25 bars + Slab bars 10 D13 bars
$M_d = 58810\ kNm$
$V_{dh} = 5660\ kN$
$M_d = 75220\ kNm$
$V_{dh} = 6080\ kN$
Vertical joint
$M_d = 92850\ kNm$
$N_d = 2309\ kN$
Vertical reinforcement 22 D25 bars
Horizontal reinforcement 28 D13 bars
$V_{dh} = 6430\ kN$
$M_d = 111500\ kNm$
8 shear keys
D13 bar $\sigma_y = 295 MPa$
D25 bar $\sigma_y = 345 MPa$
Concrete $f_c = 36 MPa$
w_i =width of a shear key (150mm)
x_i =length of a shear key at its bottom (200mm)
Horizontal joint

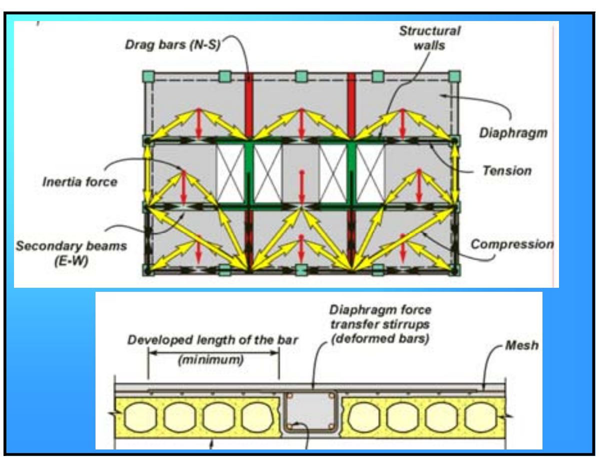

Design example of precast wall system
l_w
h_w
Compression stress field
Horizontal component of concrete compression is sustained by wall horizontal reinforcement
$V_{wh} \frac{h_w}{l_w}$
V_{wh}
Arch strut
V_{cc}
T_u
Lateral shear force carried by Truss Mechanism
Lateral shear force carried by Arch Mechanism
Horizontal shear V_{wh} is resisted by shear friction
Compressive force of arch is pull back by beam bars, a part of wall horizontal bars and slab bars

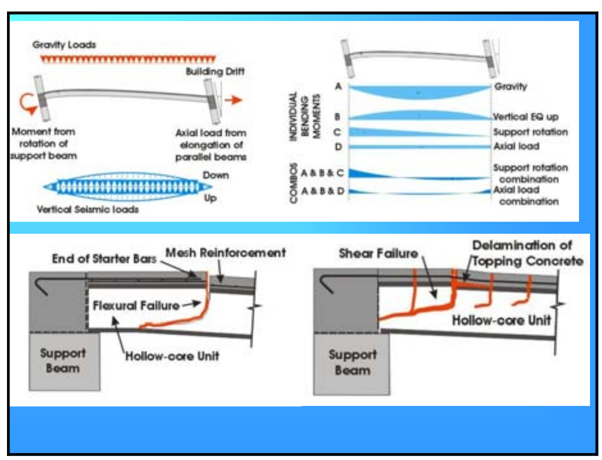

Pretensioned hollow core slab units

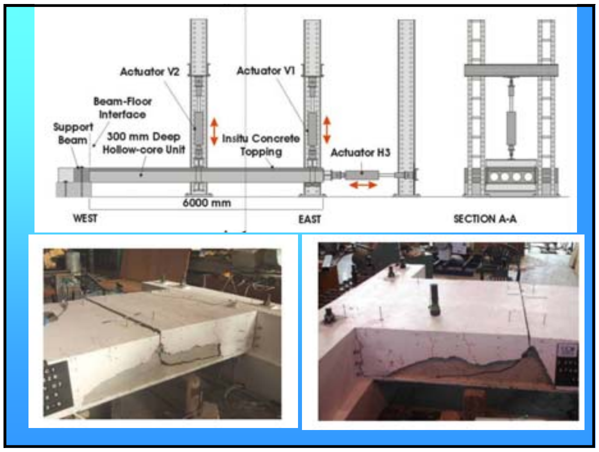

Understanding the Complexities
of Designing Diaphragms in
Buildings for Earthquakes


UNDESIRABLE
PREFERRED
Distress
Distress
Strengthened
Assumed distributed inertia force
Nominal design storey force (total inertia for the floor)
Reactions in perimeter frames
Reactions in interior frames
Reactions in interior frames
Node j
Node k
Node m
Node n
Node i
Tension
(a) Forces
(b) Moments
(c) Shear Forces
(d) Internal forces


Compression
Beams
Columns
Tension
Inertia forces
Perimeter beam
Tension in beam
Interior beams
Compression fields
Inertia force
Shear friction
Tension in slab reinforcement
Node A
Node B
PLAN
"Beam" or "Tied Arch" or "Strut & Tie" ?

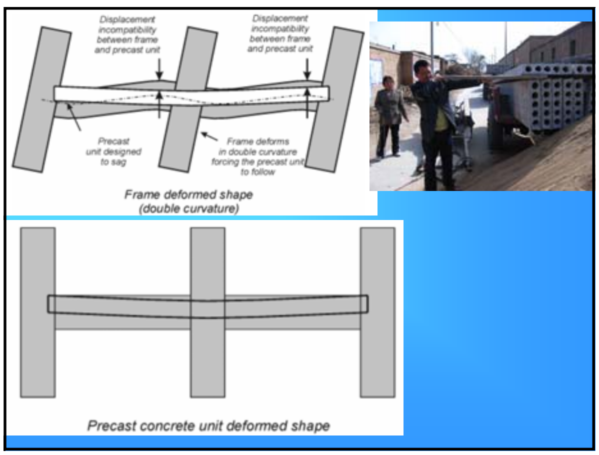

Drag bars (N-S)
Structural walls
Diaphragm
Tension
Inertia force
Secondary beams (E-W)
Compression
Diaphragm force transfer stirrups (deformed bars)
Developed length of the bar (minimum)
Mesh


Gravity Loads
Building Drift
Moment from rotation of support beam
Axial load from elongation of parallel beams
Down
Up
Vertical Seismic loads
Gravity
Vertical EQ up
Support rotation
Axial load
Support rotation combination
Axial load combination
End of Starter Bars
Mesh Reinforcement
Flexural Failure
Hollow-core Unit
Support Beam
Shear Failure
Delamination of Topping Concrete
Hollow-core Unit
Support Beam

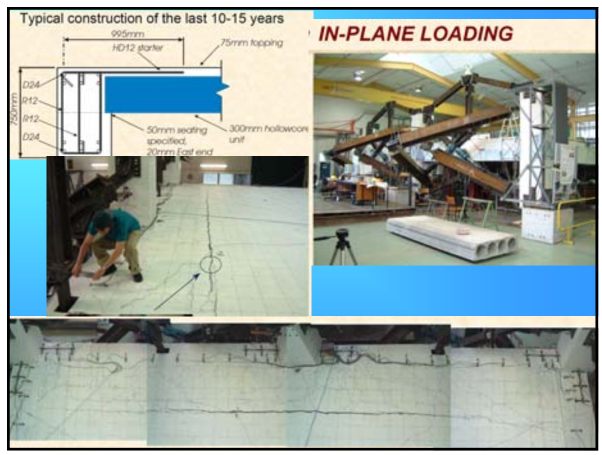


Actuator V2
Actuator V1
Beam-Floor Interface
Support Beam
300 mm Deep Hollow-core Unit
Insitu Concrete Topping
Actuator H3
6000 mm
WEST
EAST
SECTION A-A

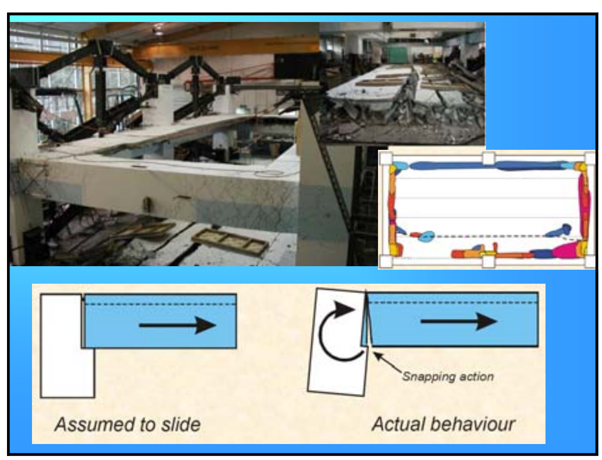
IN-PLANE LOADING
OUT-OF-PLANE LOADING



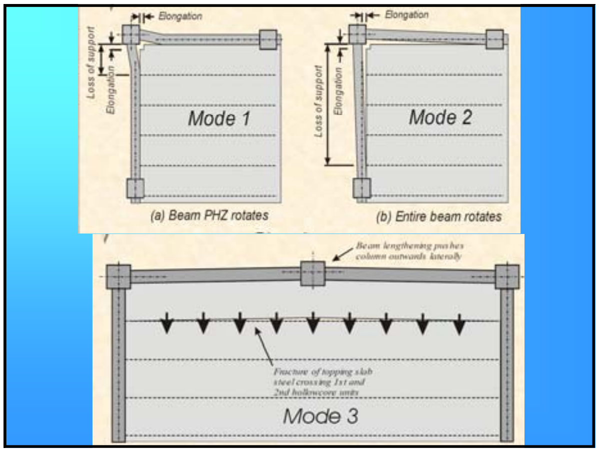
Displacement incompatibility between frame and precast unit
Displacement incompatibility between frame and precast unit
Precast unit designed to sag
Frame deforms in double curvature forcing the precast unit to follow
Frame deformed shape (double curvature)
Precast concrete unit deformed shape

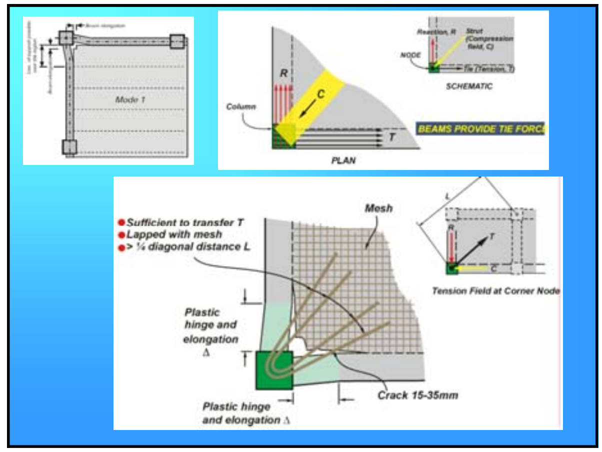


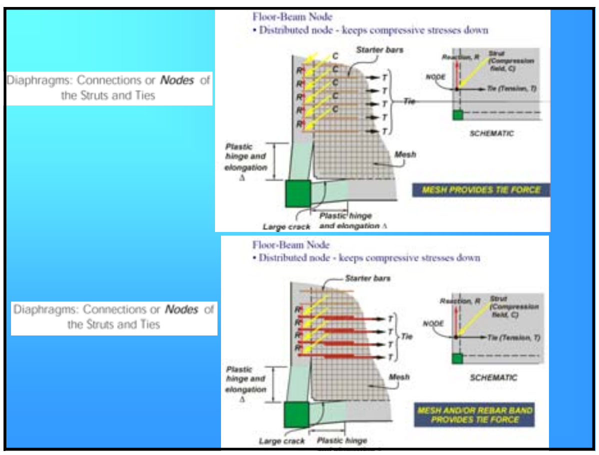
Typical construction of the last 10-15 years
995mm
HD12 starter
75mm topping
50mm seating specified, 20mm East end
300mm hollowcore unit
IN-PLANE LOADING

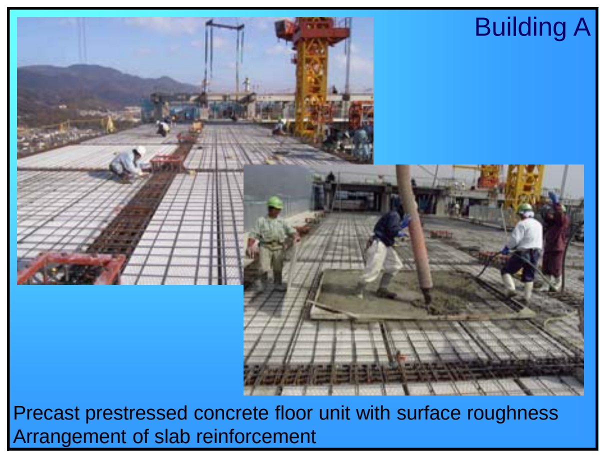


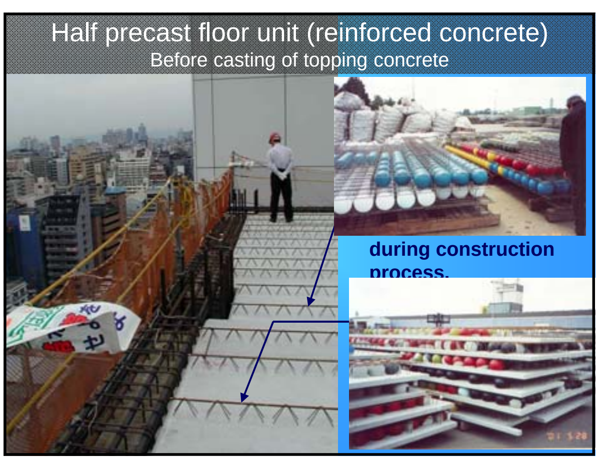
Snapping action
Assumed to slide
Actual behaviour

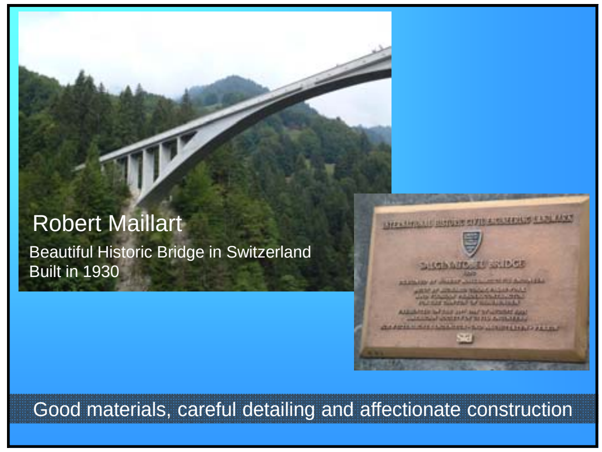

Elongation
Loss of support
Elongation
Mode 1
Elongation
Loss of support
Elongation
Mode 2
(a) Beam PHZ rotates
(b) Entire beam rotates
Beam lengthening pushes column outwards laterally
Fracture of topping slab steel crossing 1st and 2nd hollowcore units
Mode 3


Mode 1
Column
R
C
T
PLAN
Reaction, R
Strut (Compression field, C)
NODE
Tie (Tension, T)
SCHEMATIC
BEAMS PROVIDE TIE FORC
Sufficient to transfer T
Lapped with mesh
> ¼ diagonal distance L
Mesh
Tension Field at Corner Node
Plastic hinge and elongation Δ
Crack 15-35mm
Plastic hinge and elongation Δ


Diaphragms: Connections or Nodes of the Struts and Ties
Floor-Beam Node
• Distributed node - keeps compressive stresses down
Starter bars
Reaction, R
Strut (Compression field, C)
NODE
Tie (Tension, T)
Tie
SCHEMATIC
Plastic hinge and elongation Δ
Mesh
MESH PROVIDES TIE FORCE
Large crack
Plastic hinge and elongation Δ
Diaphragms: Connections or Nodes of the Struts and Ties
Floor-Beam Node
• Distributed node - keeps compressive stresses down
Starter bars
Reaction, R
Strut (Compression field, C)
NODE
Tie
Tie (Tension, T)
Plastic hinge and elongation Δ
Mesh
SCHEMATIC
MESH AND/OR REBAR BAND PROVIDES TIE FORCE
Large crack
Plastic hinge

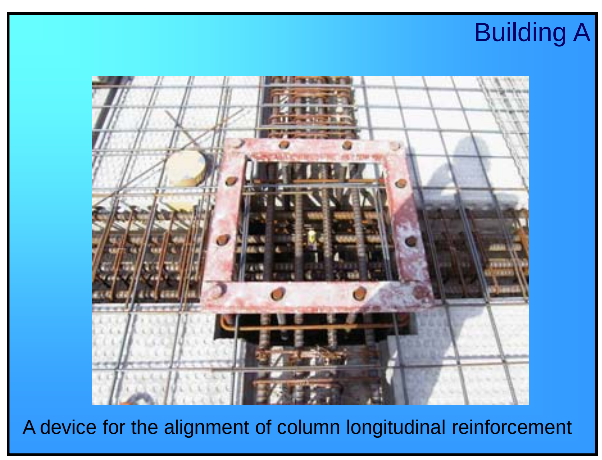

Building A
Precast prestressed concrete floor unit with surface roughness
Arrangement of slab reinforcement


Half precast floor unit (reinforced concrete)
Before casting of topping concrete
during construction process.

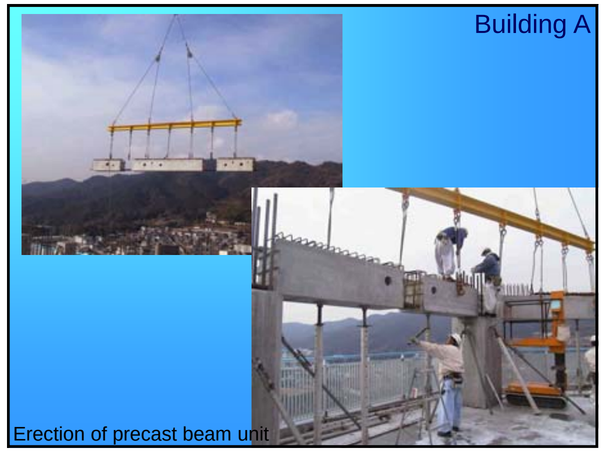

Robert Maillart
Beautiful Historic Bridge in Switzerland
Built in 1930
Good materials, careful detailing and affectionate construction


Actual Construction of Precast Concrete Buildings

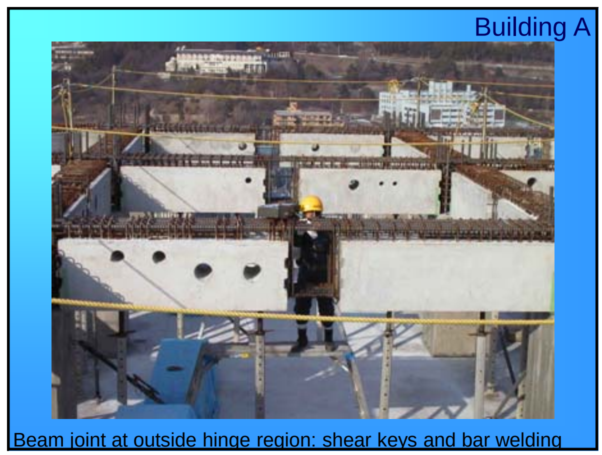

Precast reinforced concrete moment frame
Building A
38 storied condominium building, Building height 130.7m

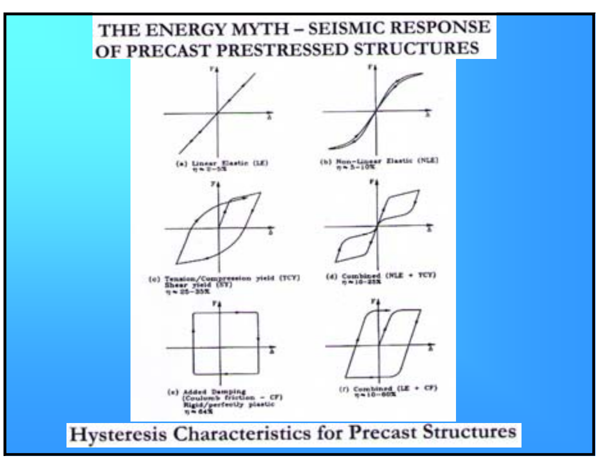

Building A
A device for the alignment of column longitudinal reinforcement

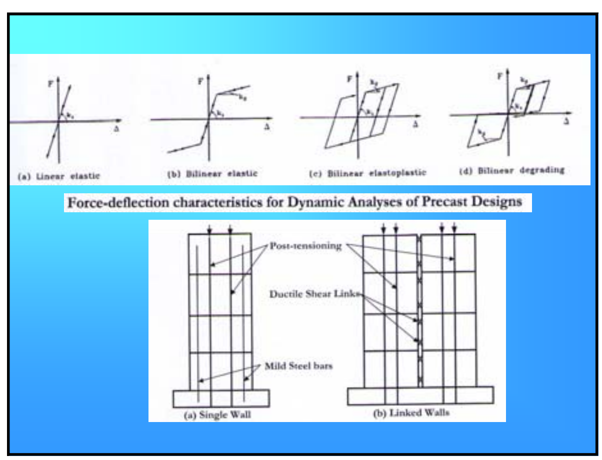

Building A
Erection of column unit
Mortar injection to mechanical coupler and column base gap

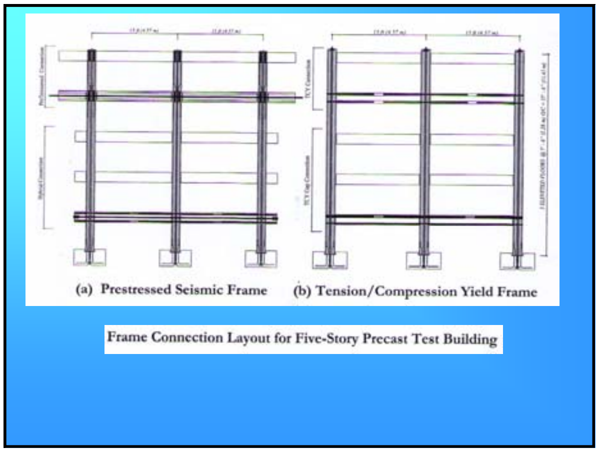

Building A
Erection of precast beam unit

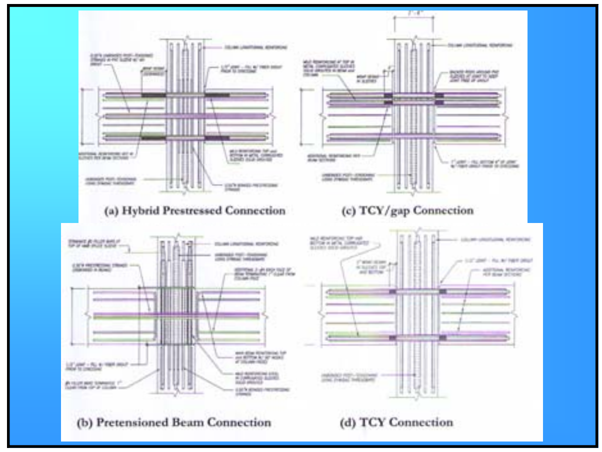

Building A
Beam column joint


Building A
Beam joint at outside hinge region: shear keys and bar welding

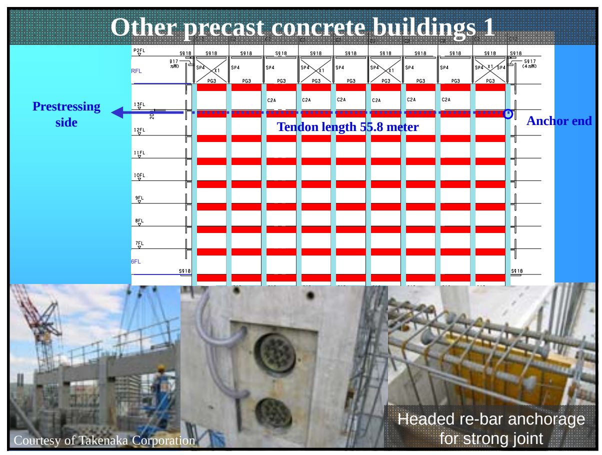

THE ENERGY MYTH – SEISMIC RESPONSE OF PRECAST PRESTRESSED STRUCTURES
(a) Linear Elastic (LE)
(b) Non-Linear Elastic (NLE)
(c) Tension/Compression yield (TCY) Shear yield (SY)
(d) Combined (NLE + TCY)
(e) Added Damping (Coulomb friction – CF) Rigid/perfectly plastic
(f) Combined (LE + CF)
Hysteresis Characteristics for Precast Structures


(a) Linear elastic
(b) Bilinear elastic
(c) Bilinear elastoplastic
(d) Bilinear degrading
Force-deflection characteristics for Dynamic Analyses of Precast Designs
Post-tensioning
Ductile Shear Links
Mild Steel bars
(a) Single Wall
(b) Linked Walls


(a) Prestressed Seismic Frame
(b) Tension/Compression Yield Frame
Frame Connection Layout for Five-Story Precast Test Building


(a) Hybrid Prestressed Connection
(c) TCY/gap Connection
(b) Pretensioned Beam Connection
(d) TCY Connection


Other precast concrete buildings 1
Courtesy of Takenaka Corporation

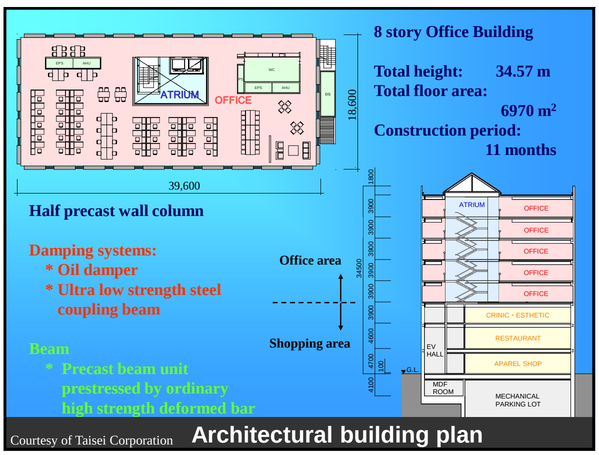

Other precast concrete buildings 1
Prestressing side
Tendon length 55.8 meter
Anchor end
Headed re-bar anchorage for strong joint
Courtesy of Takenaka Corporation

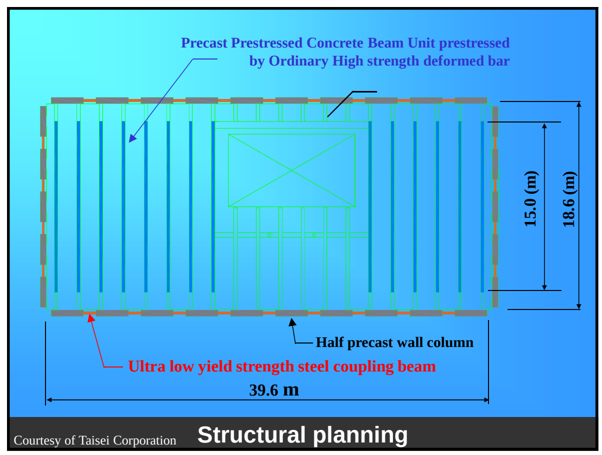

Other precast concrete buildings 2
Beam bottom face
Erection
Beam top face
Courtesy of Takenaka Corporation

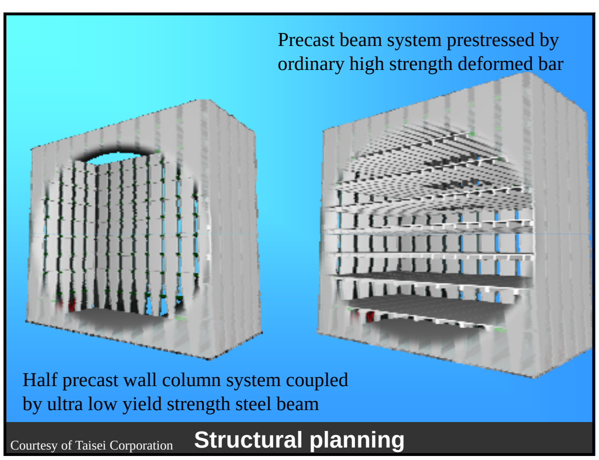

Other precast concrete buildings 2
Lap splice of wall horizontal reinforcement protruded from precast column unit
Cast in place wall concrete
Headed wall horizontal bar anchorage
After casting of slab topping concrete
Courtesy of Takenaka Corporation

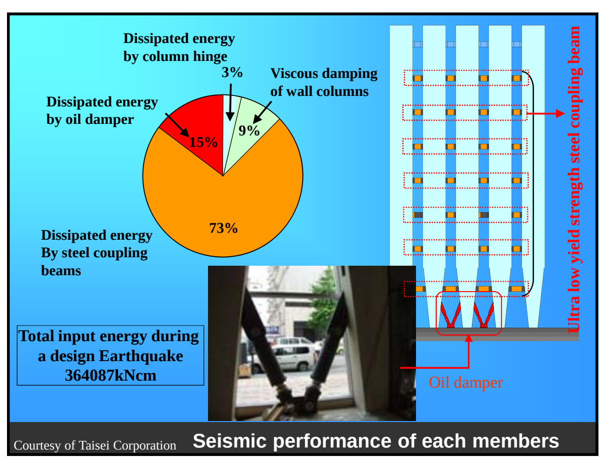

Other precast concrete buildings 3, 4
Detailing
Beam unit
Beam column joint unit
Diagonally reinforced column unit
High rise apartment building
Courtesy of Takenaka Corporation

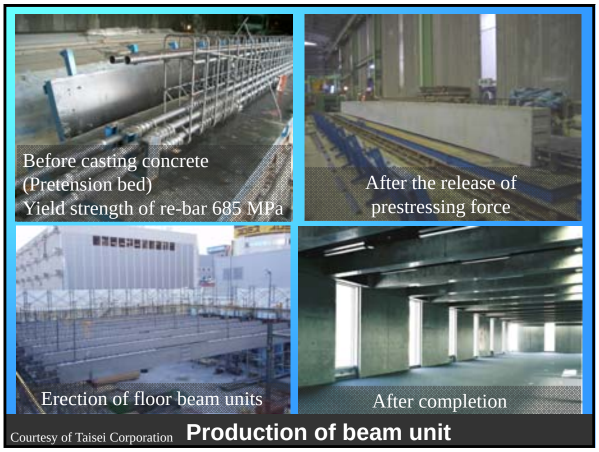

Office building in Hokkaido
(Precast concrete + Damping system)
8 story Office Building
Total height: 34.57 m
Total floor area: 6970 m²
Construction period: 11 months
Courtesy of Taisei Corporation


8 story Office Building
Total height: 34.57 m
Total floor area: 6970 m²
Construction period: 11 months
ATRIUM
OFFICE
39,600
18,600
Half precast wall column
Damping systems:
* Oil damper
* Ultra low strength steel coupling beam
Beam
* Precast beam unit prestressed by ordinary high strength deformed bar
Office area
Shopping area
CRINIC・ESTHETIC
RESTAURANT
APAREL SHOP
EV HALL
MDF ROOM
MECHANICAL PARKING LOT
Courtesy of Taisei Corporation
Architectural building plan

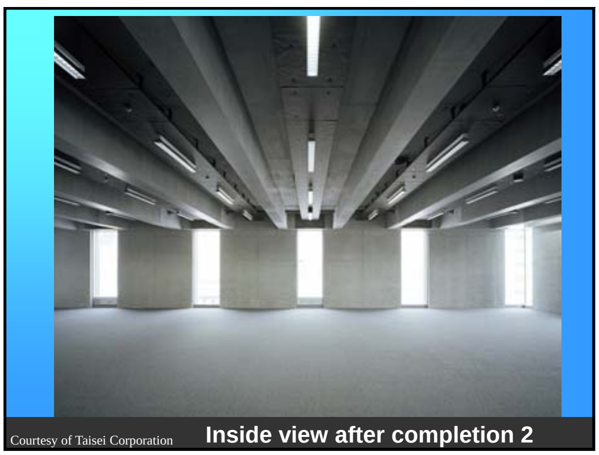

Precast Prestressed Concrete Beam Unit prestressed by Ordinary High strength deformed bar
15.0 (m)
18.6 (m)
Half precast wall column
Ultra low yield strength steel coupling beam
39.6 m
Courtesy of Taisei Corporation
Structural planning


Precast beam system prestressed by ordinary high strength deformed bar
Half precast wall column system coupled by ultra low yield strength steel beam
Courtesy of Taisei Corporation
Structural planning

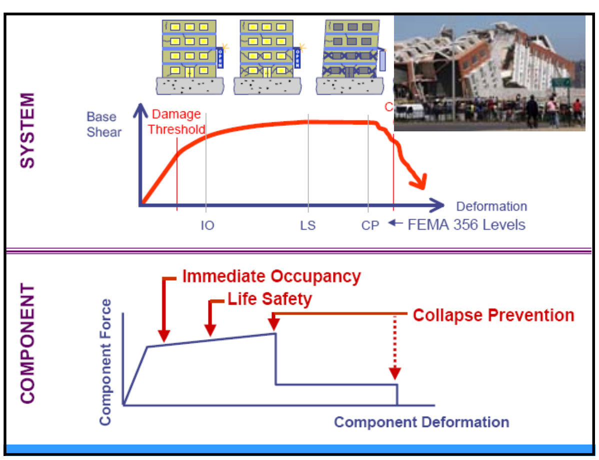

Dissipated energy by column hinge
3%
Viscous damping of wall columns
9%
Dissipated energy by oil damper
15%
73%
Dissipated energy By steel coupling beams
Total input energy during a design Earthquake 364087kNcm
Ultra low yield strength steel coupling beam
Oil damper
Courtesy of Taisei Corporation
Seismic performance of each members

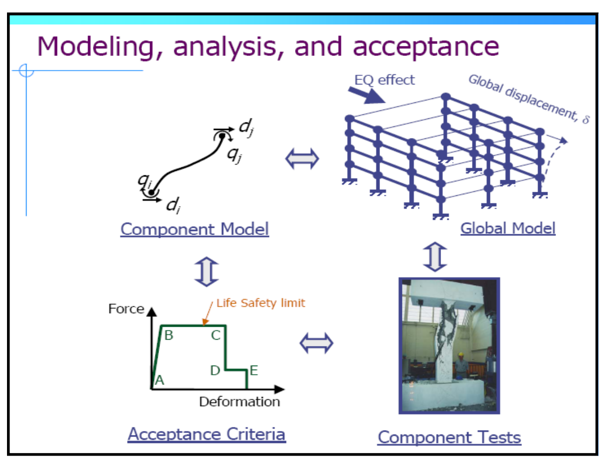

Before casting concrete
(Pretension bed)
Yield strength of re-bar 685 MPa
After the release of
prestressing force
Erection of floor beam units
After completion
Courtesy of Taisei Corporation
Production of beam unit

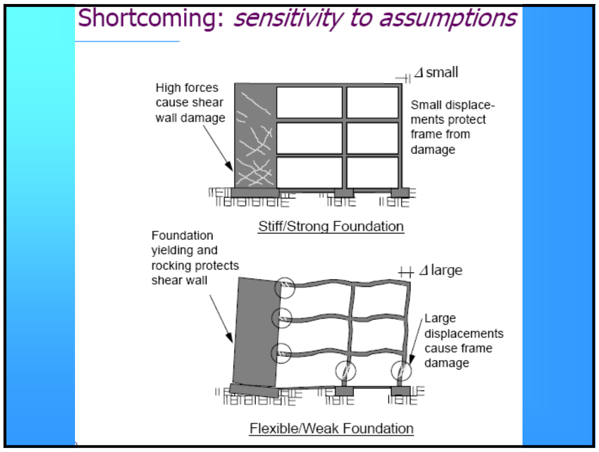

Courtesy of Taisei Corporation
Inside view after completion 1


Courtesy of Taisei Corporation
Inside view after completion 2


Concentrated damage to coupling steel beams during the earthquake
Replaceable
No or slight damage to concrete wall columns and precast prestressed beams during the earthquake
Continuous building service
High cost performance due to controlled response
Cost competitive
Realized performances in this building
Courtesy of Taisei Corporation

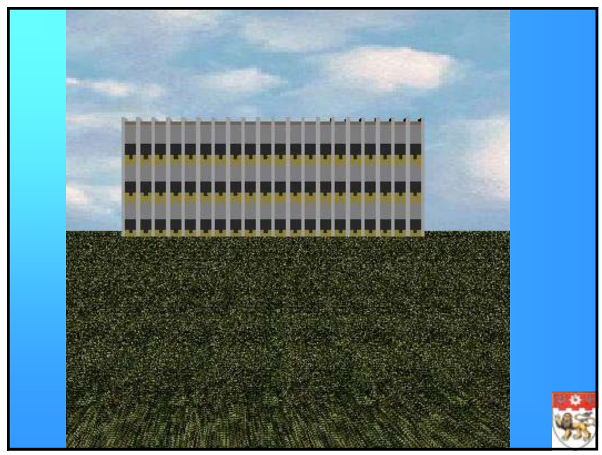

SYSTEM
Base
Shear
Damage
Threshold
Deformation
IO
LS
CP
FEMA 356 Levels
COMPONENT
Immediate Occupancy
Life Safety
Collapse Prevention
Component Force
Component Deformation

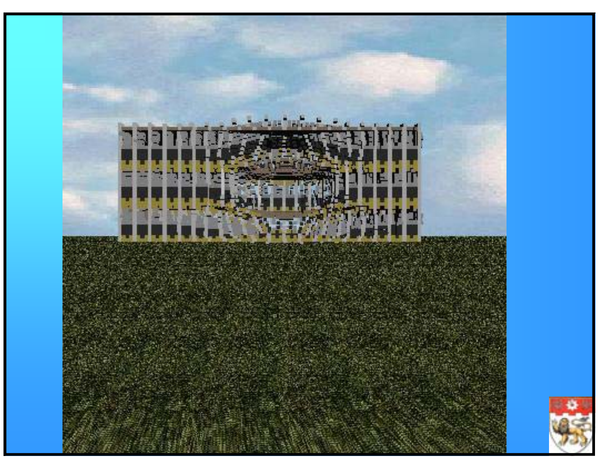

Modeling, analysis, and acceptance
EQ effect
Global displacement, δ
Component Model
Global Model
Force
Life Safety limit
Deformation
Acceptance Criteria
Component Tests

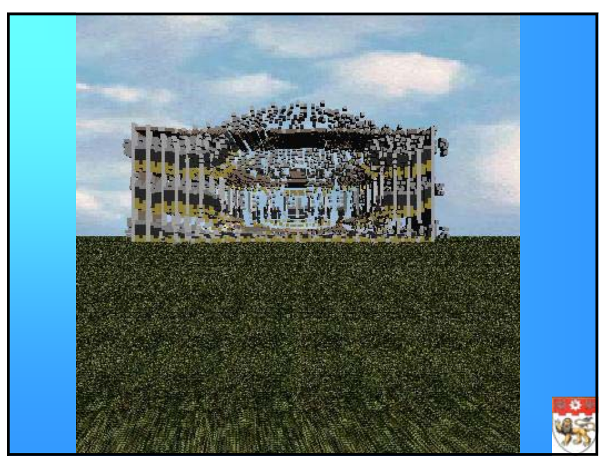

Shortcoming: sensitivity to assumptions
High forces cause shear wall damage
Δ small
Small displace-ments protect frame from damage
Stiff/Strong Foundation
Foundation yielding and rocking protects shear wall
Δ large
Large displacements cause frame damage
Flexible/Weak Foundation

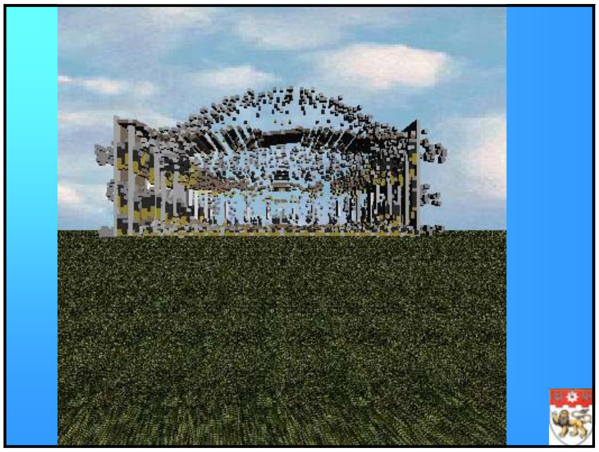

We Will Not Forget
9.11.01

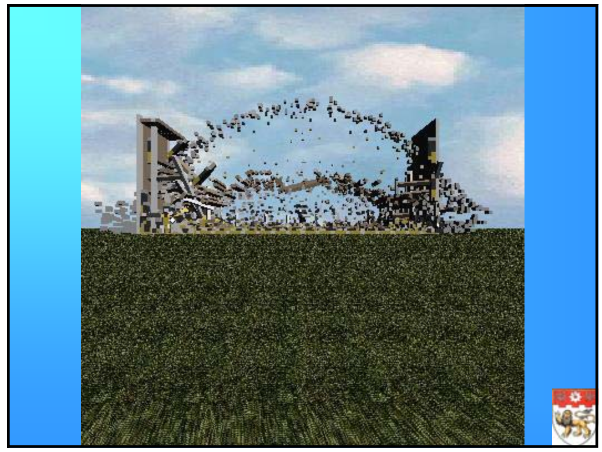

DYNAMIC PRESSURE
NORMAL PRESSURE
SHOCK FRONT












NTU Garden Campus
Thank you