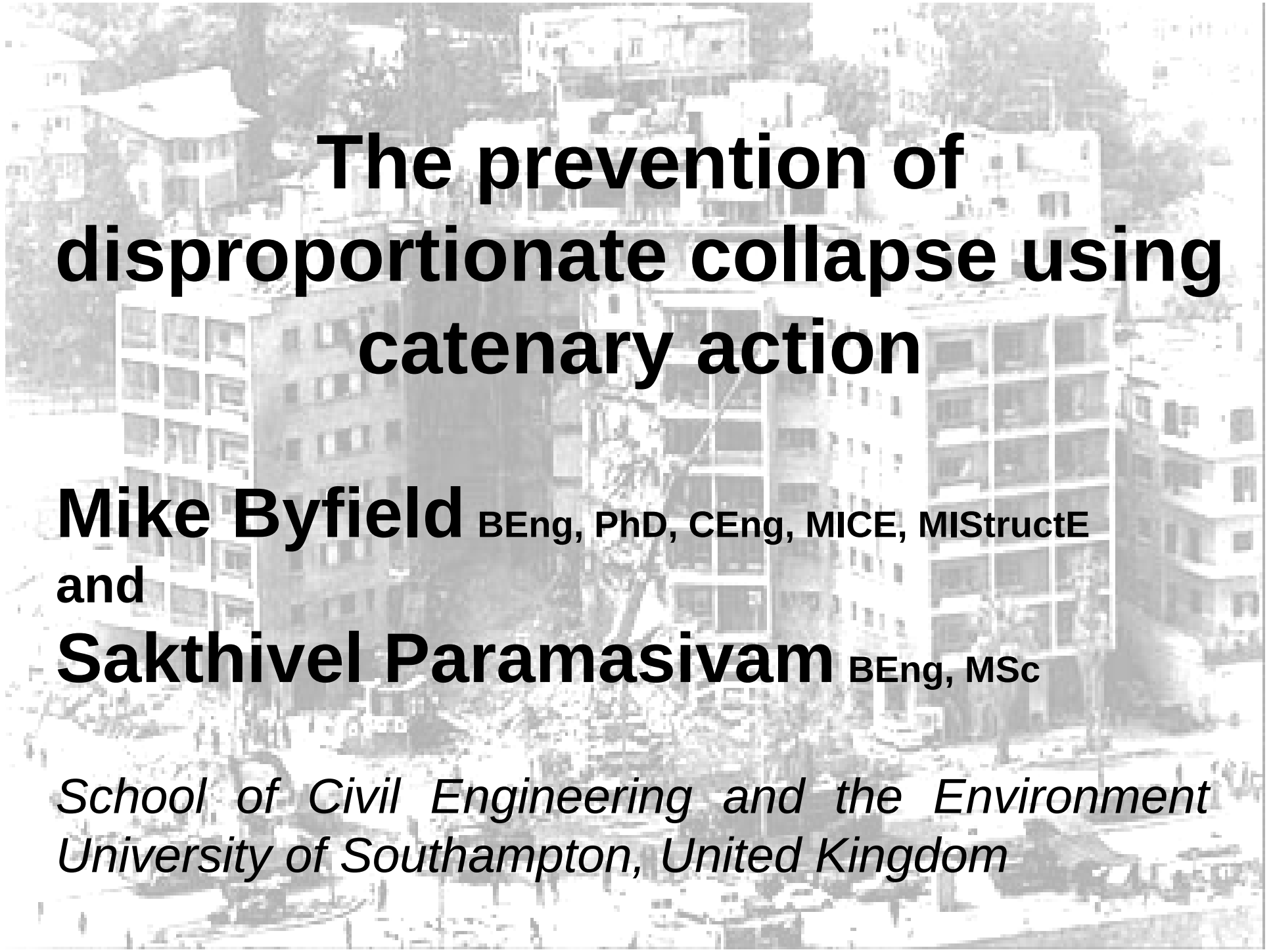

The prevention of disproportionate collapse using catenary action

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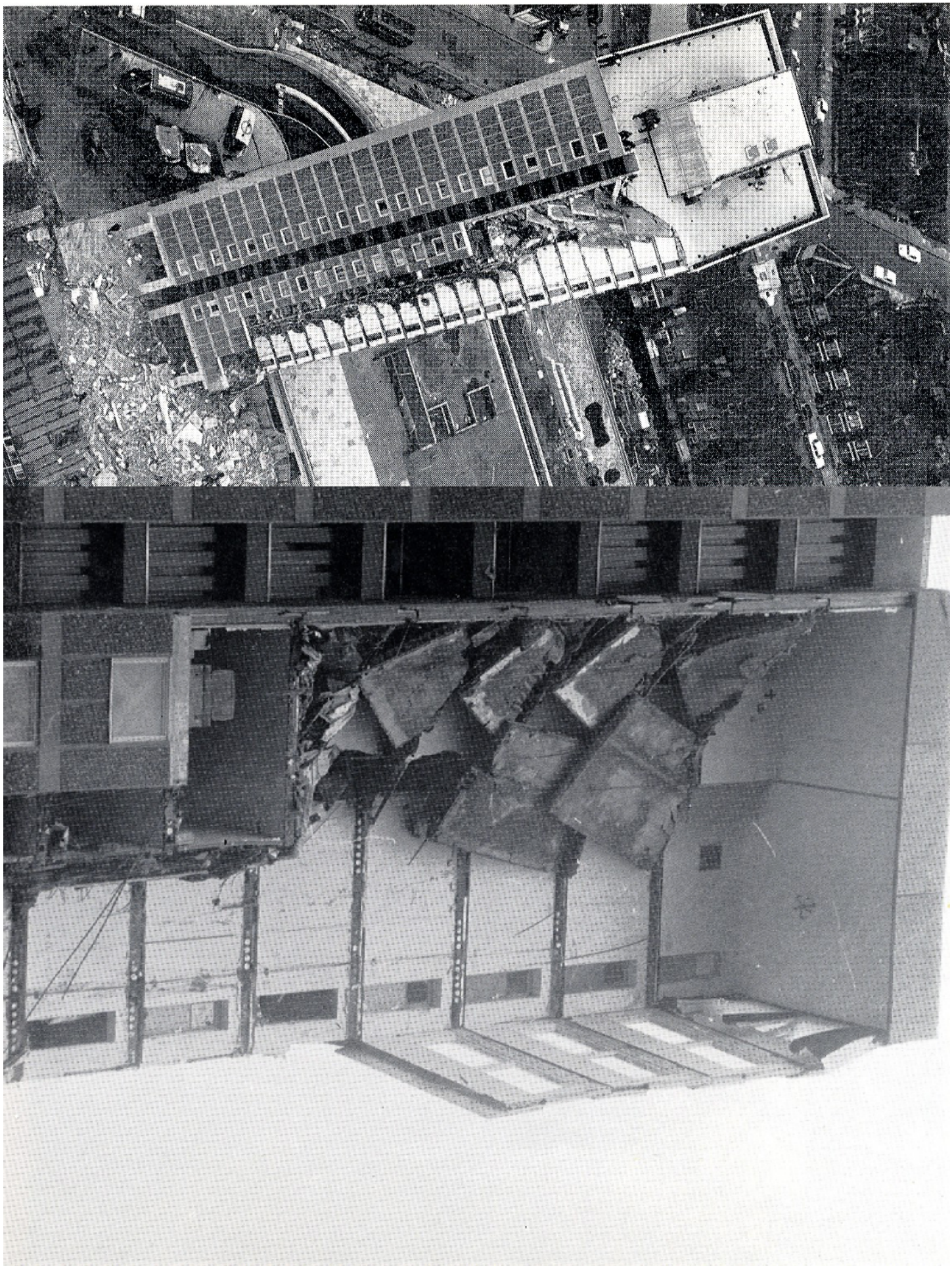
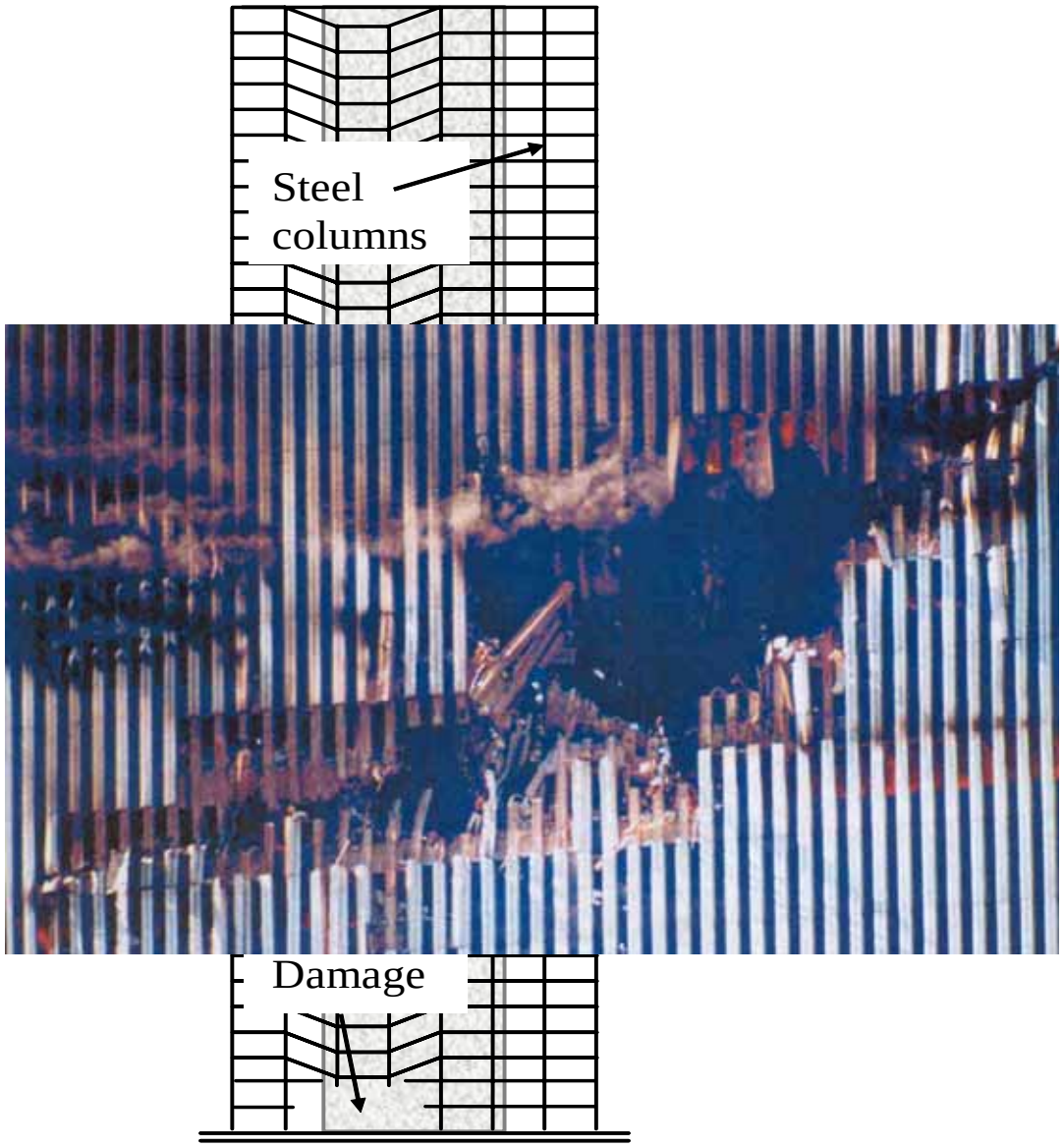
and
Sakthivel Paramasivam **BEng, MSc**

School of Civil Engineering and the Environment
University of Southampton, United Kingdom

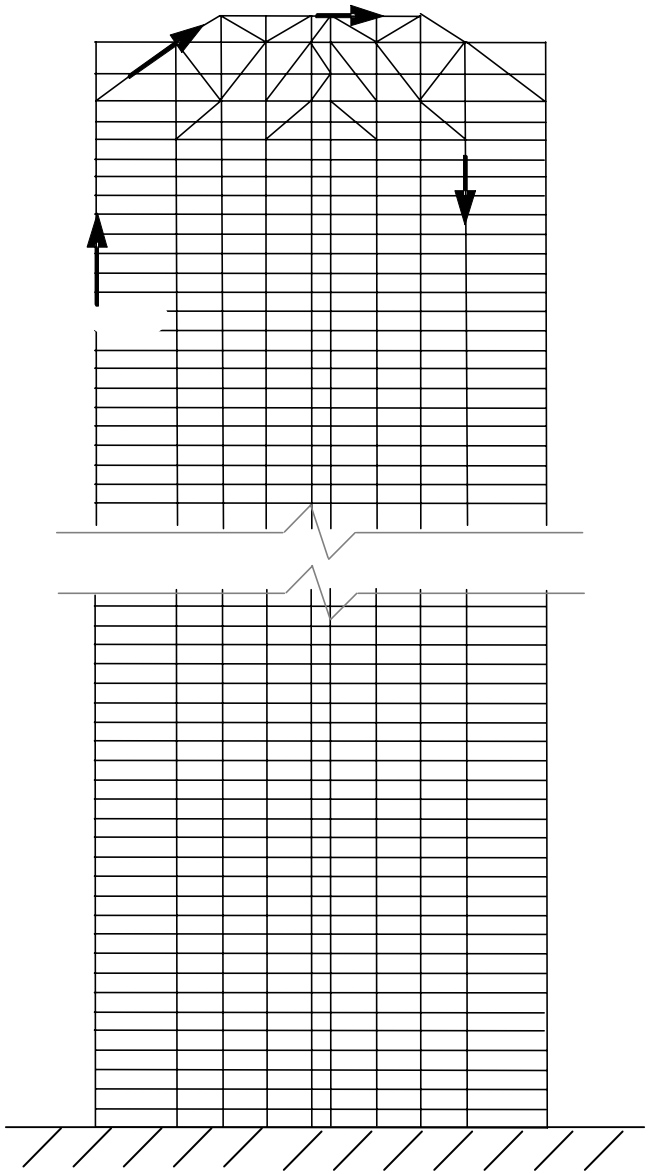


1983 US Marine Corps HQ, Lebanon - 241 dead + 60 wounded

1995 Federal Murrah Building



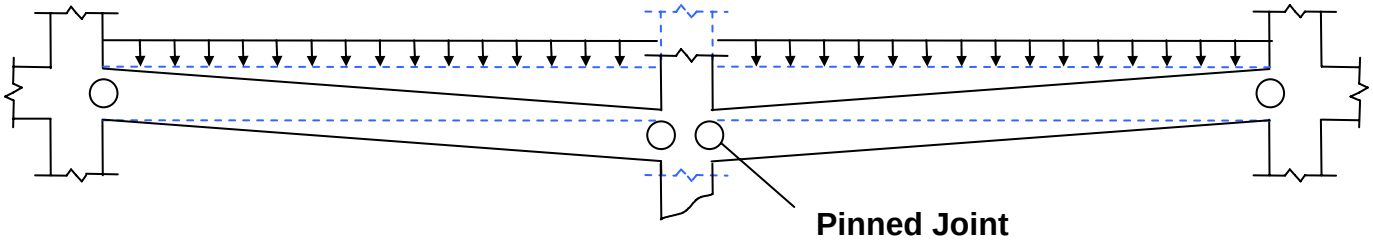
Catenary Action



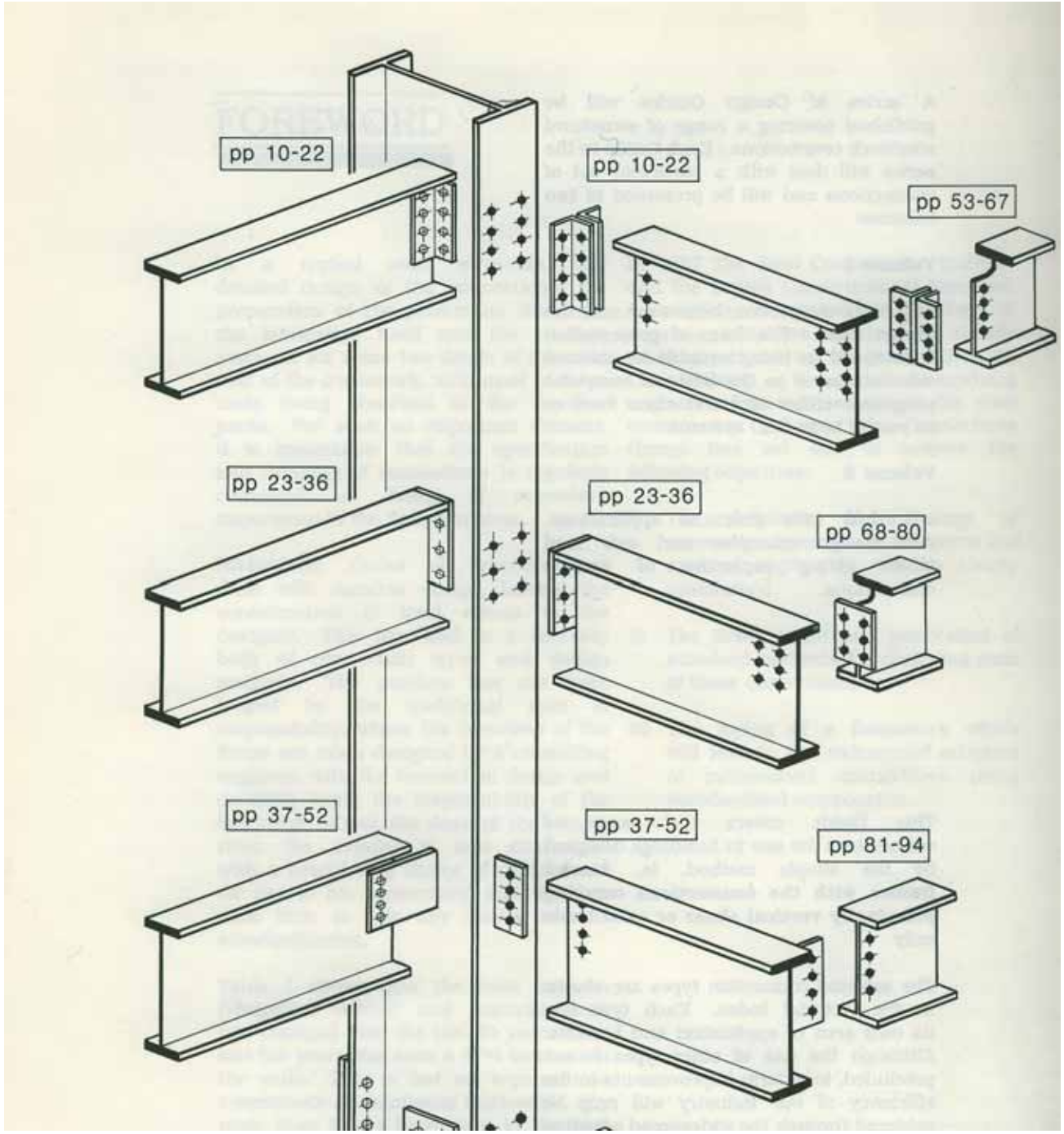
Redistribution of perimeter column loads through hat truss in WTC1

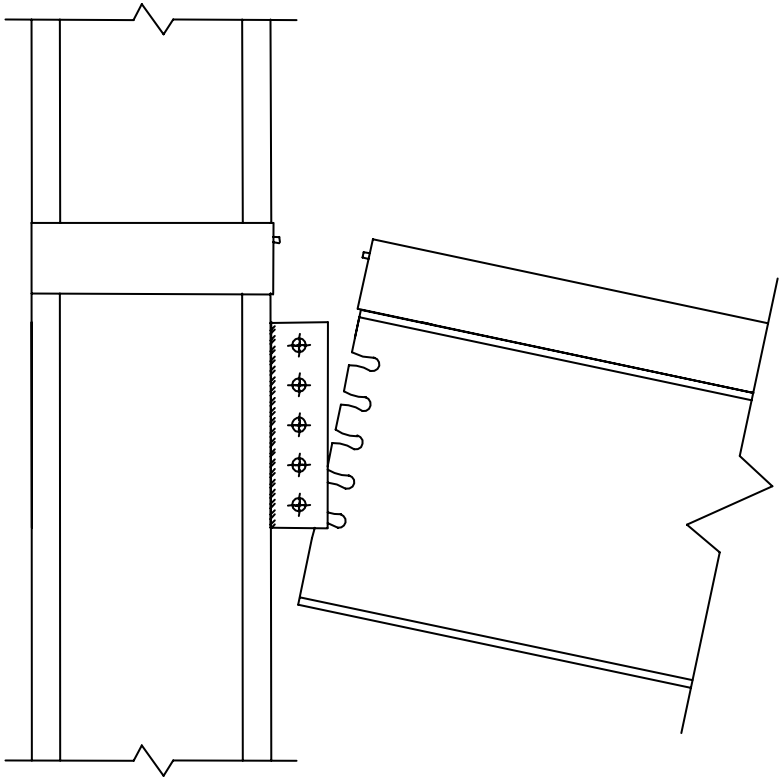
Tying Force Method

Accidental limit state load = 1.05 g_k + 0.33 q_k

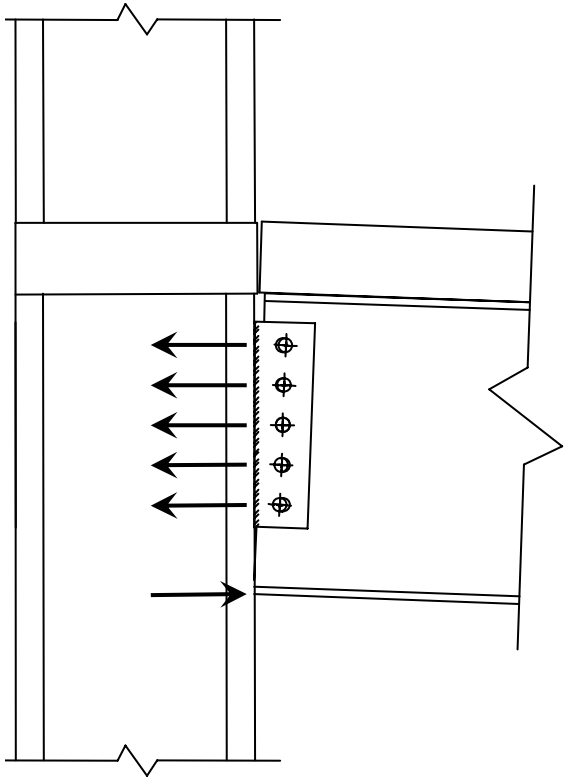


UK approach
DAF = 1
Pinned joints
Full reliance on catenary action

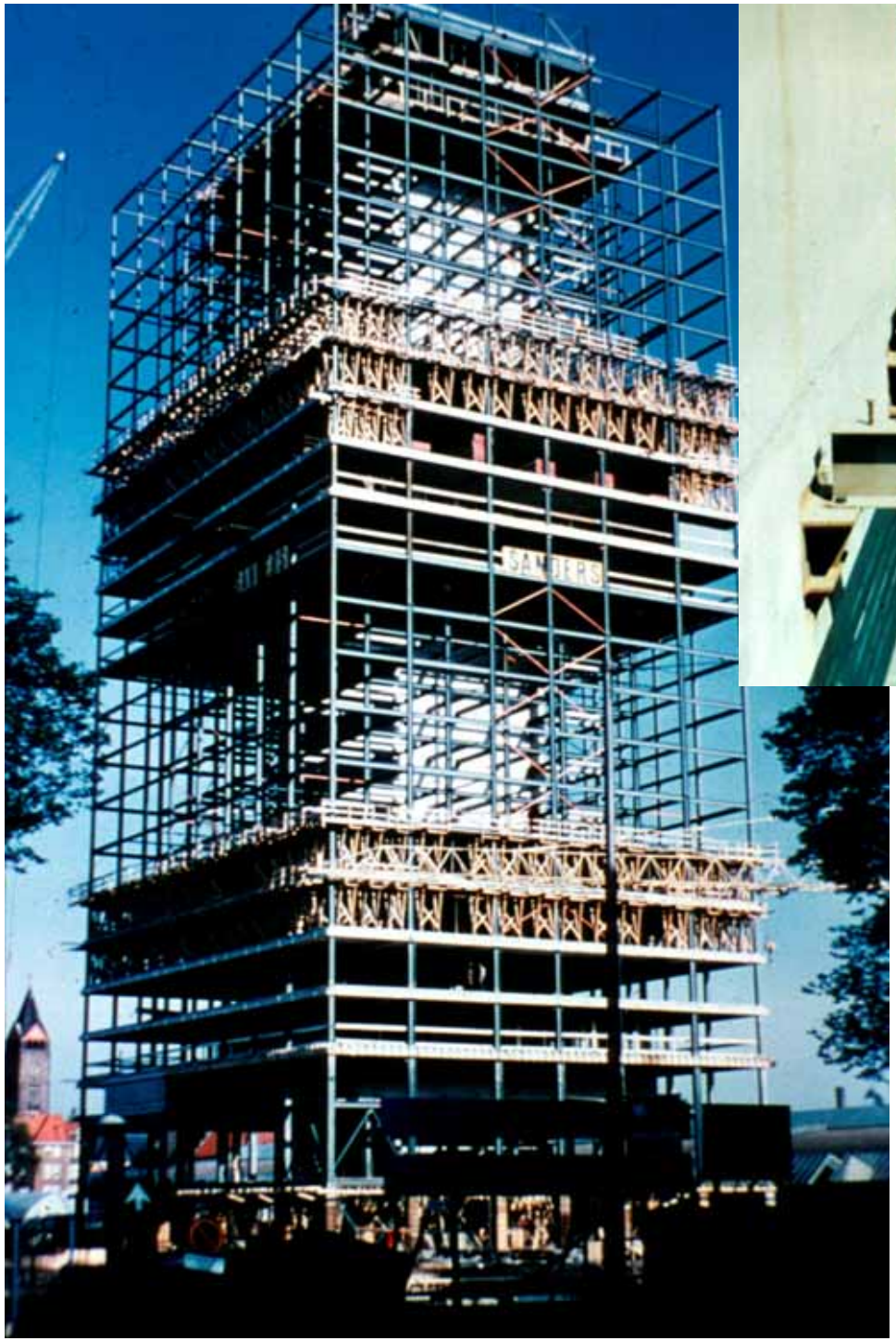

pp 10-22
pp 10-22
pp 53-67
pp 23-36
pp 23-36
pp 68-80
pp 37-52
pp 37-52
pp 81-94

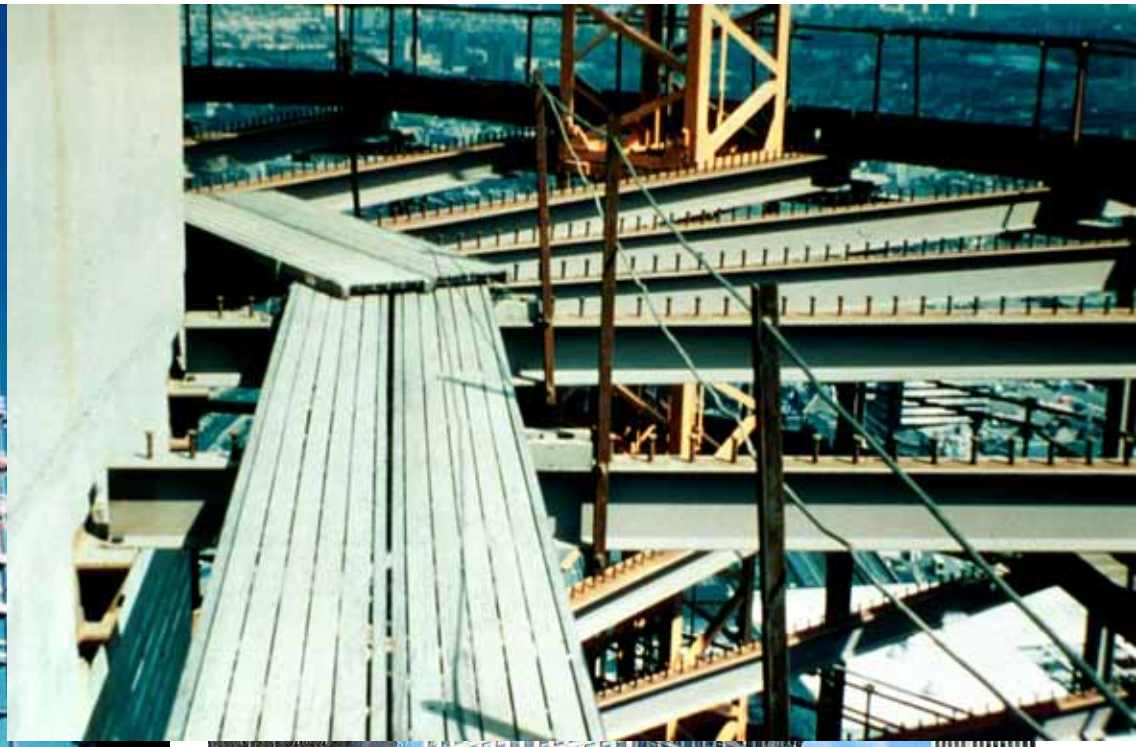


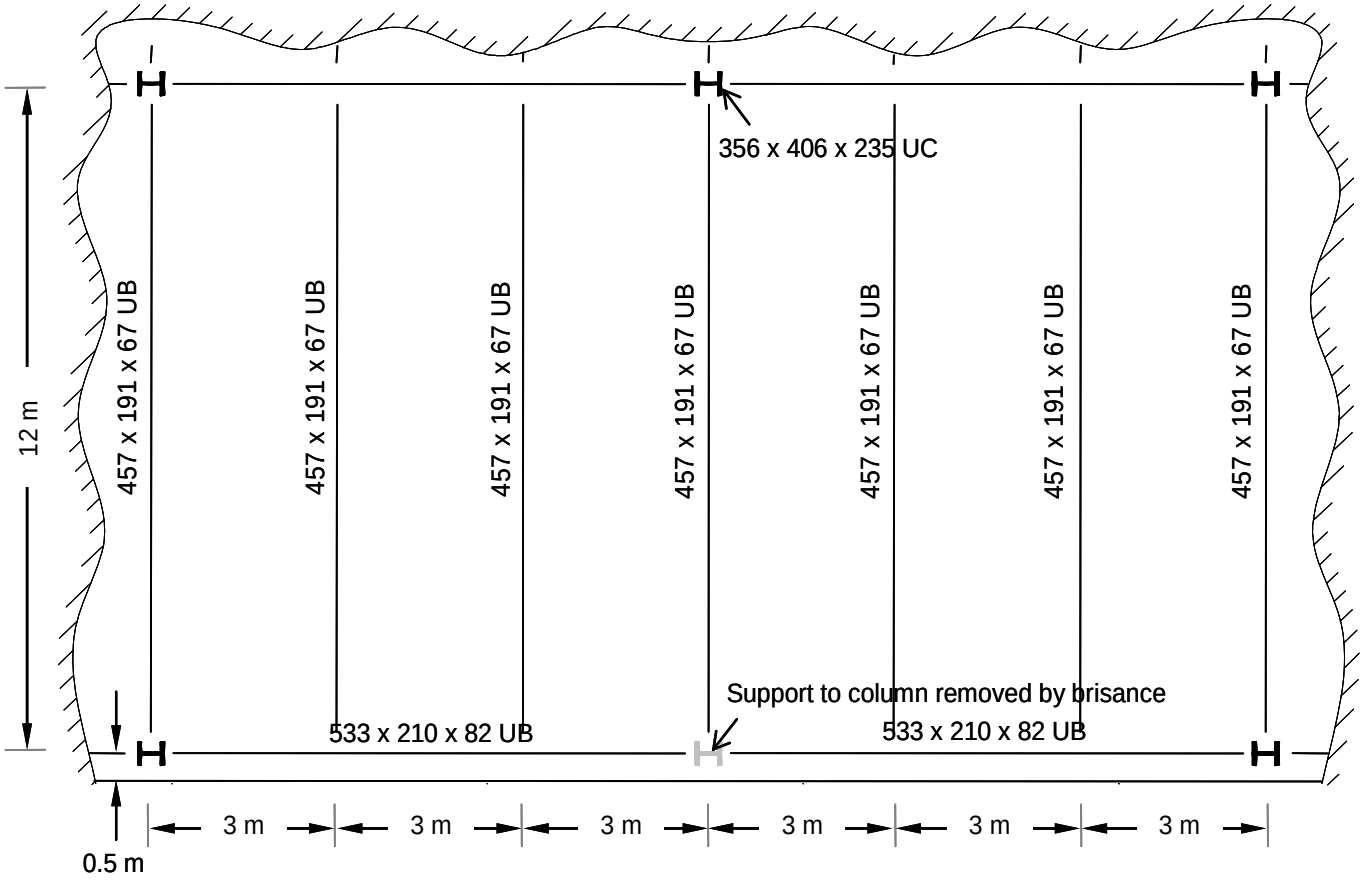
Rupture



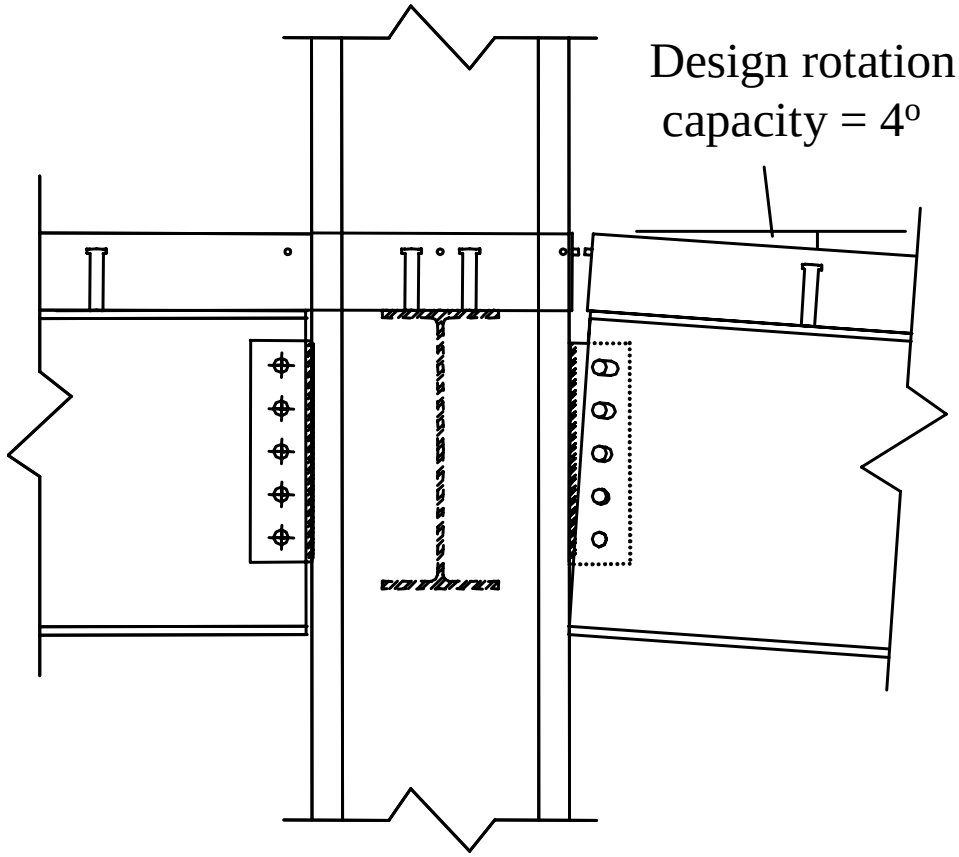
Catenary load







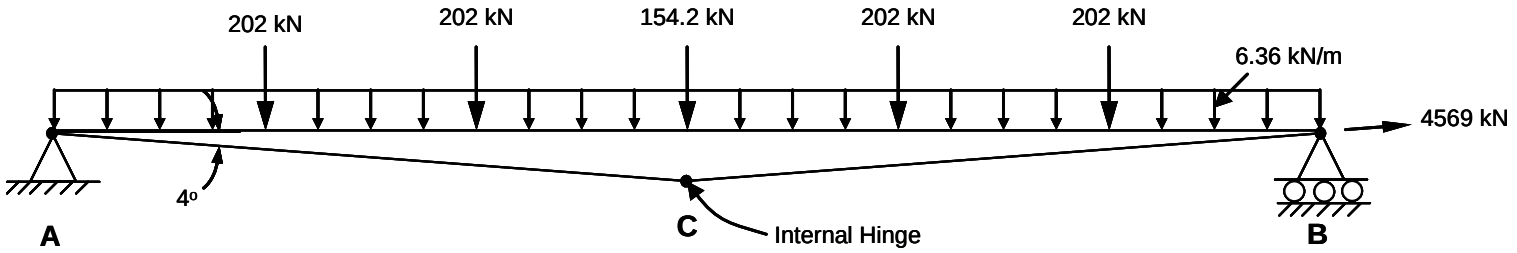
Notes:
All columns – 356 x 406 x 235 UC
All main beams – 533 x 210 x 82 UB
All secondary beam – 457 x 191 x 67 UB
Steel grade – S355
Concrete grade – C35
Imposed load – 5 kN/m^2
Partition load – 1 kN/m^2


Design rotation
capacity = 4º

The Best Guess Scenario, FoS = 0.12

- Full tensile strength of the slab included
- DAF = 1.5

Accidental limit state load = 1.05 g_k + 0.33 q_k



$$FoS = \frac{528}{4569} = 0.12$$

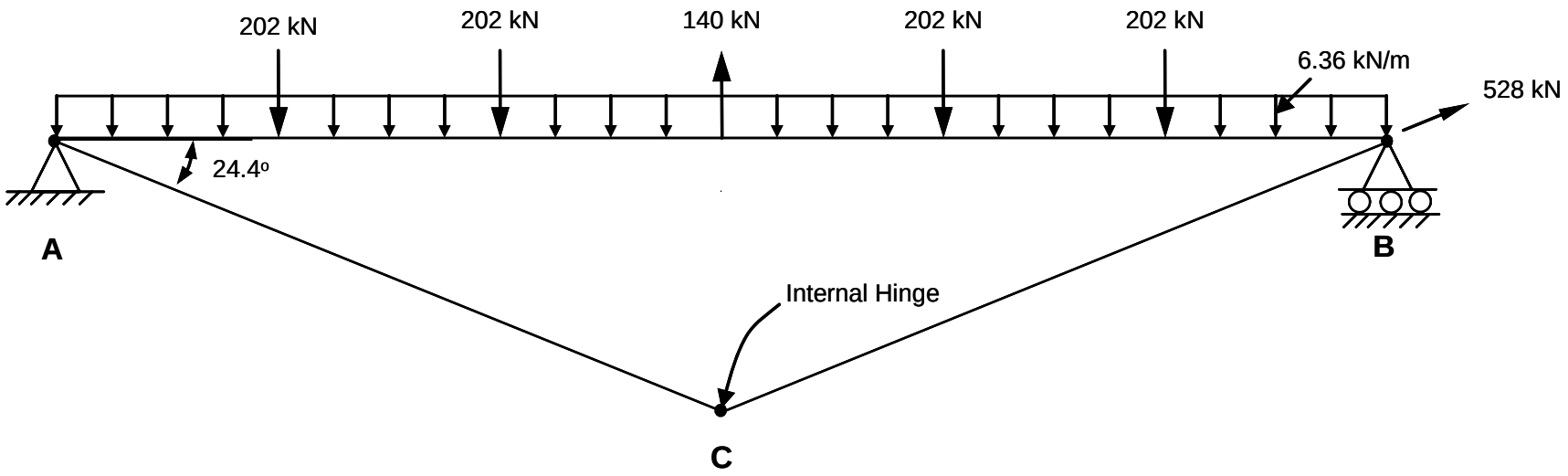
The Best Case Scenario, FoS = 0.19

- Full tensile strength of the slab included
- DAF = 1.0

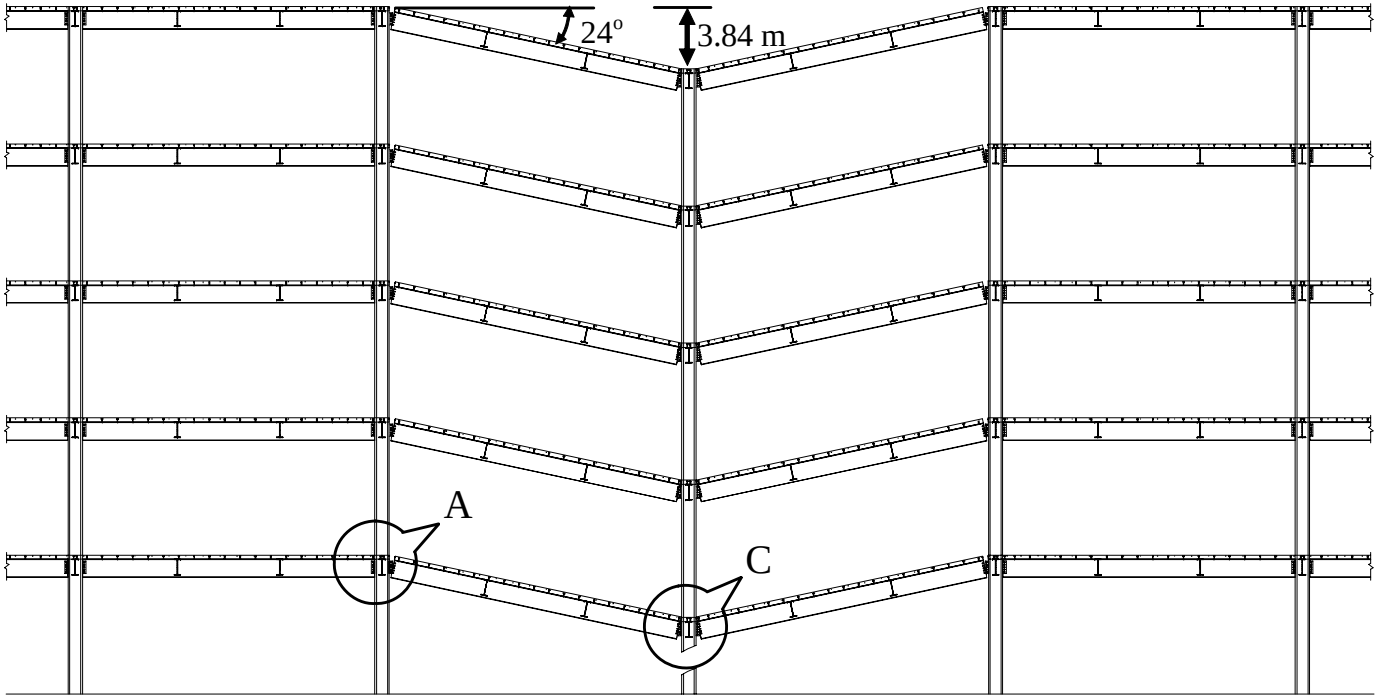
The Worst Case Scenario, FoS = 0.08

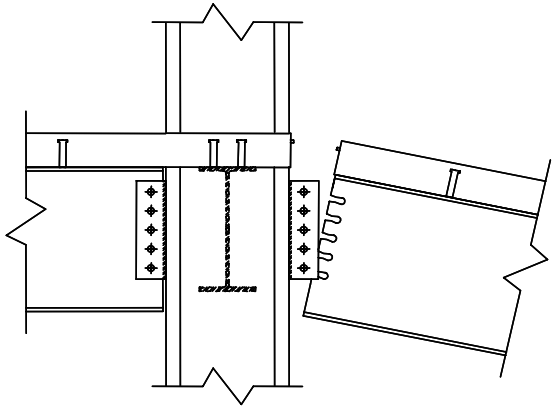
- Tensile strength of the slab ignored
- DAF = 2.0 in accordance with US practice

What if we have unlimited ductility in the connections?

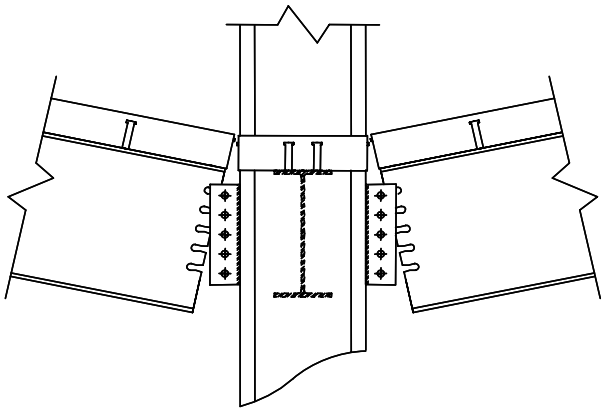


DLF=1.5
Slab strength included



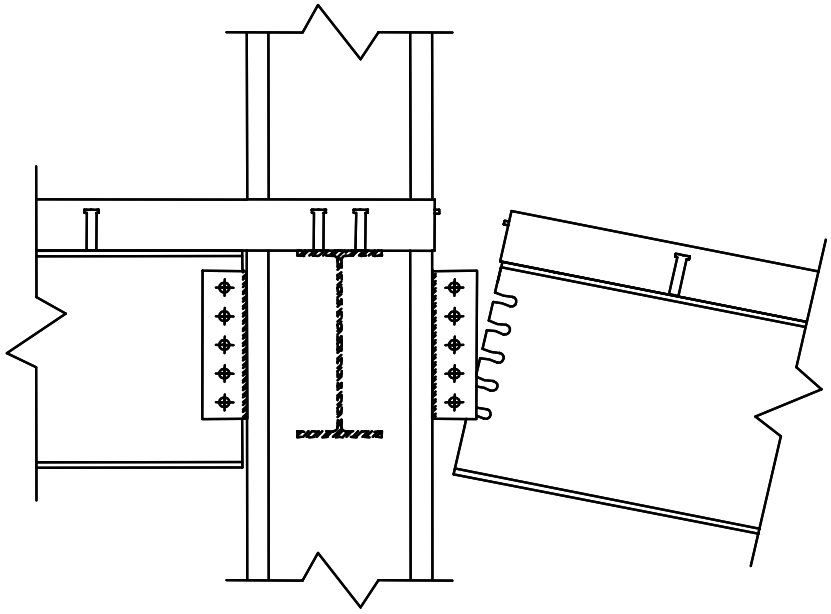


Joint ‘A’

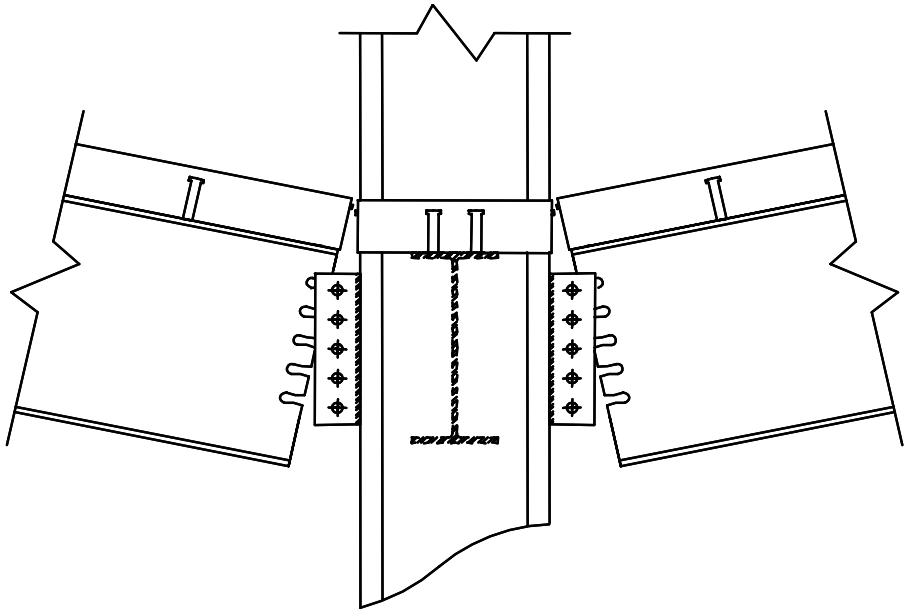


Joint ‘C’

DLF=1.5
Slab strength included

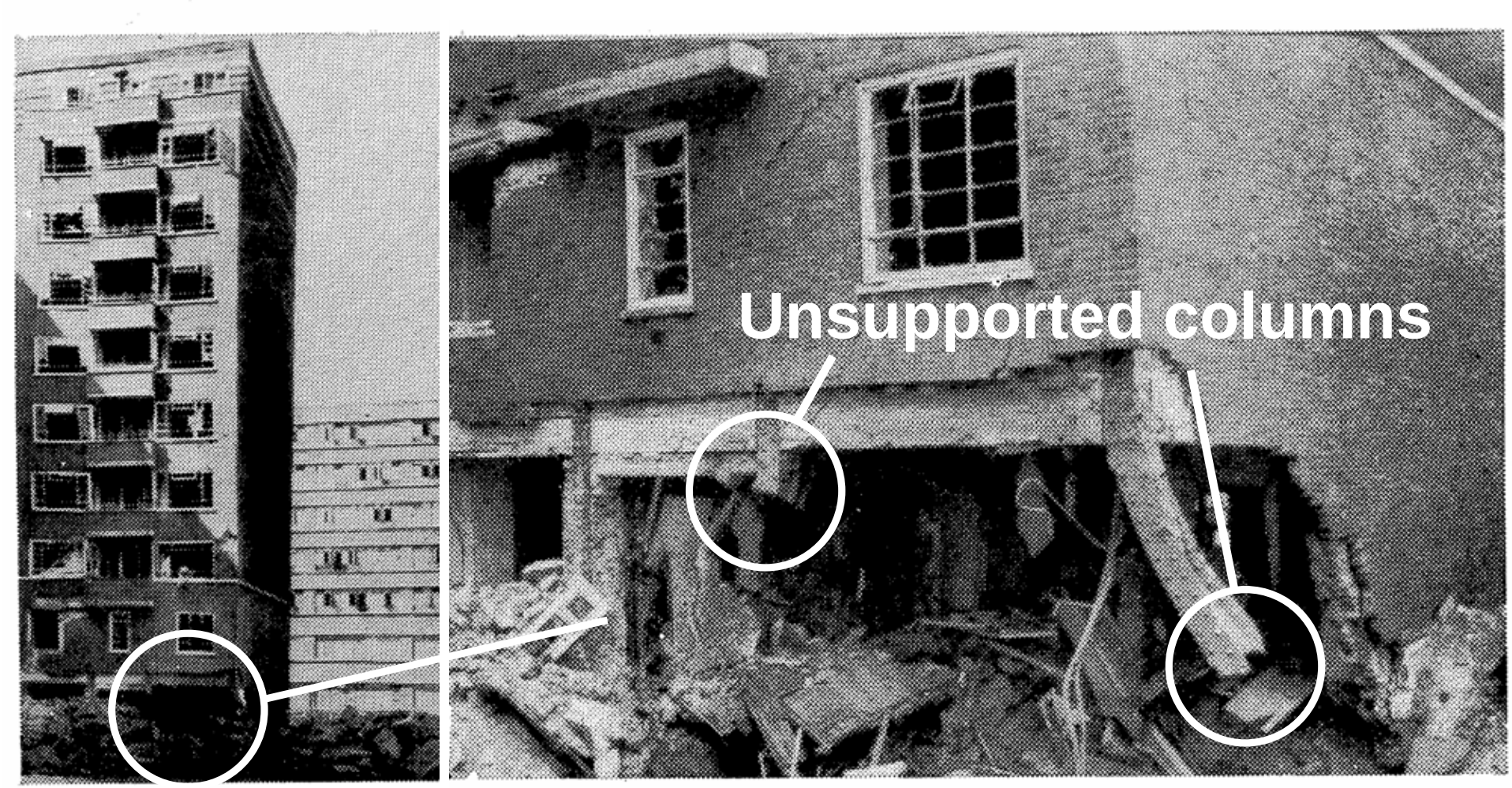


(b) Joint ‘A’



(c) Joint ‘C’

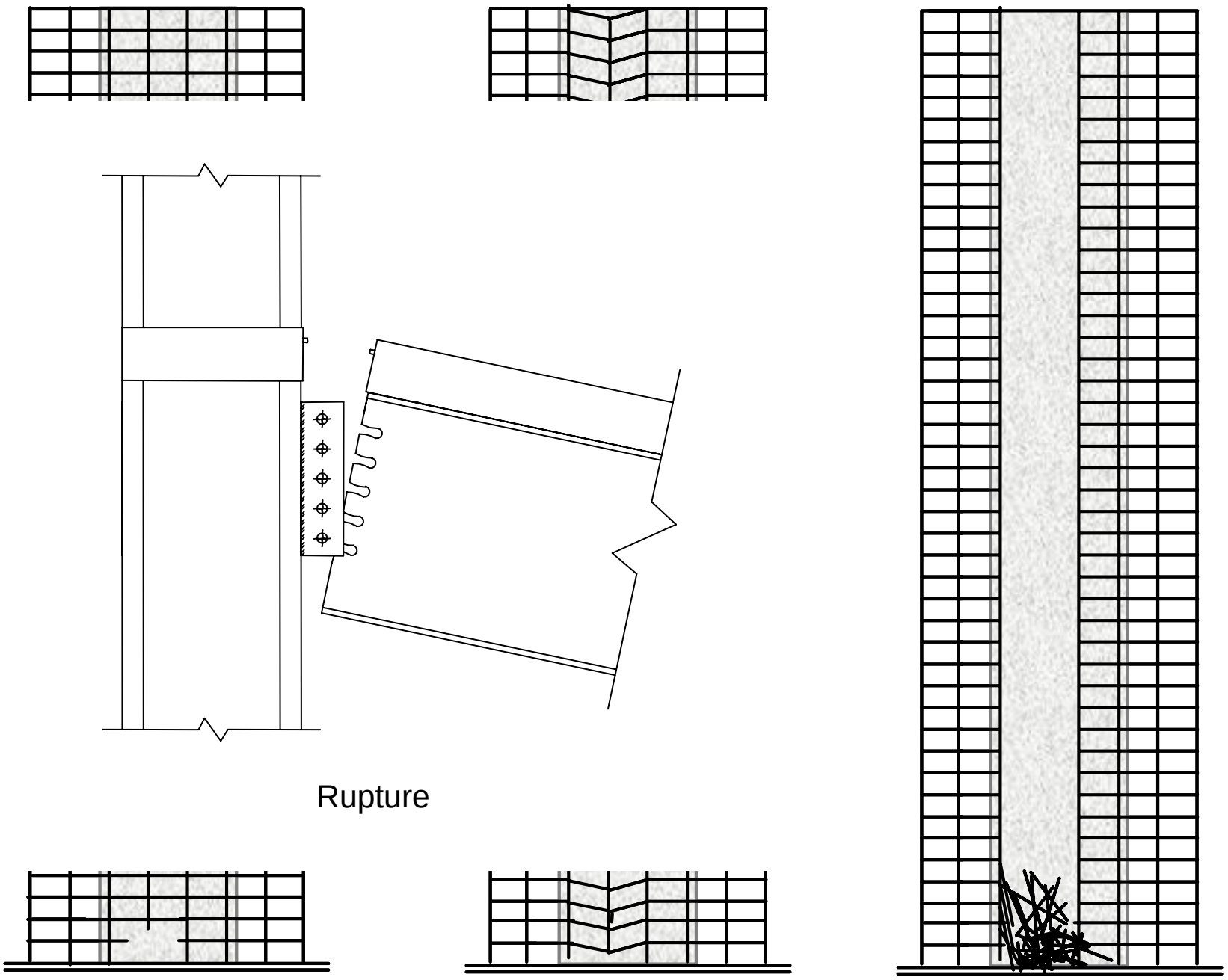

Unsupported columns


Unsupported columns

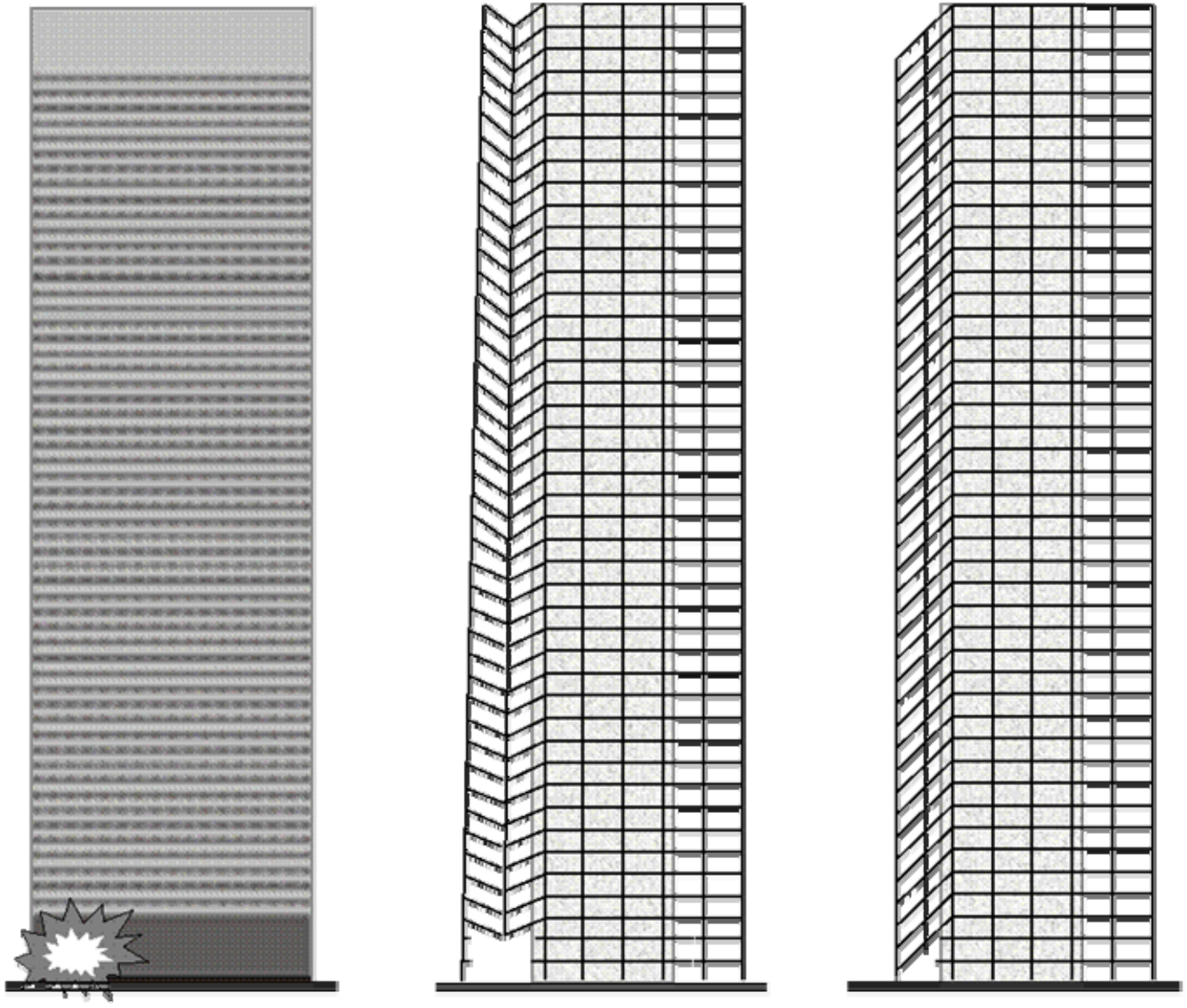
1995 Federal Murrah Building

Typical Canary Wharf Tower –

- Flexible cladding
- No stiff internal partitions
- No columns between service cores and perimeter
- Number of columns minimised by use of transfer beams
- Low stiffness slab
- Low ductility “Pinned connections”



Rupture



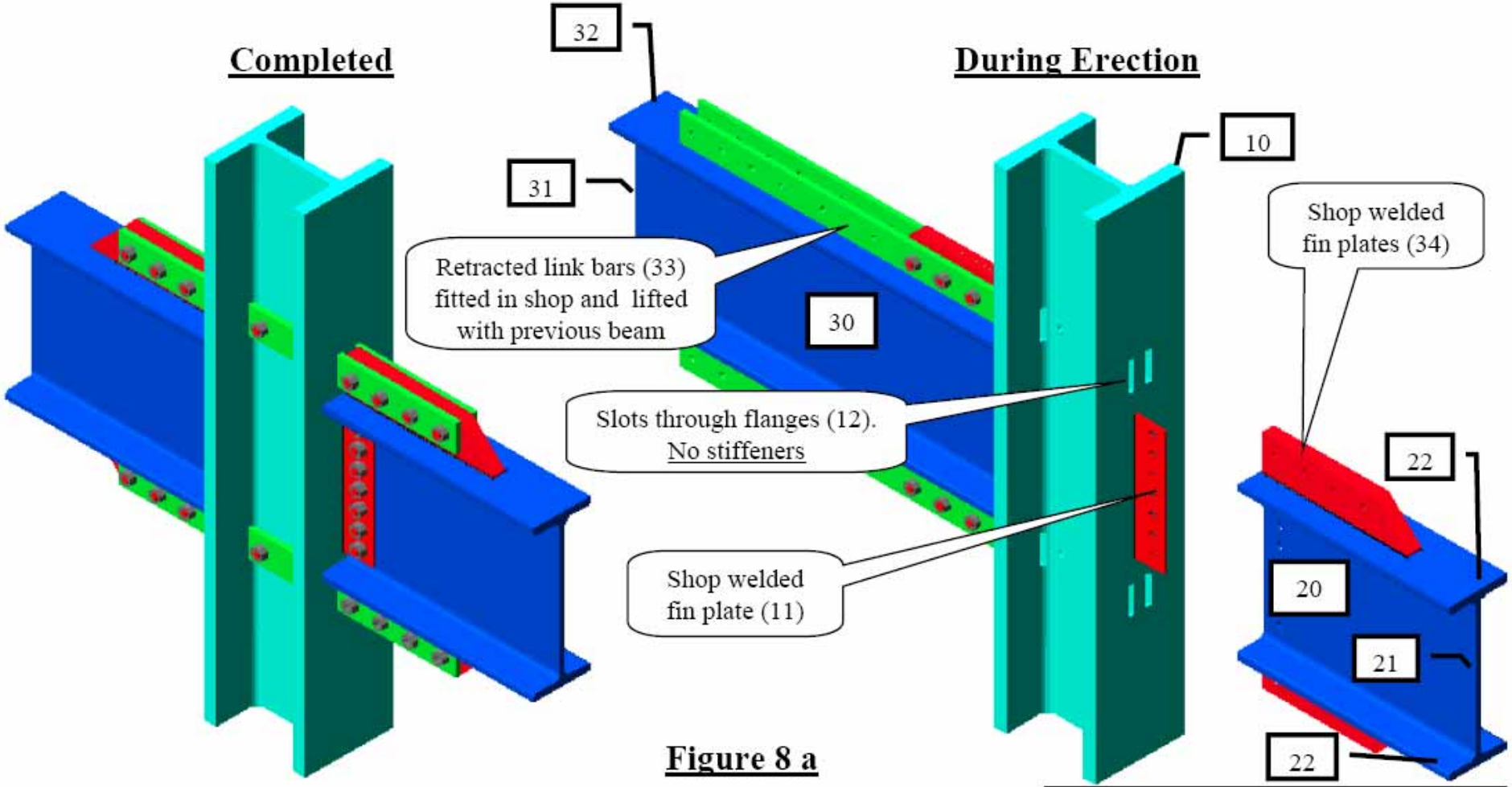


Figure 8 a

Connection – Type 4

VBH Patent

Extreme Event Beam Link Connection

ERECTION

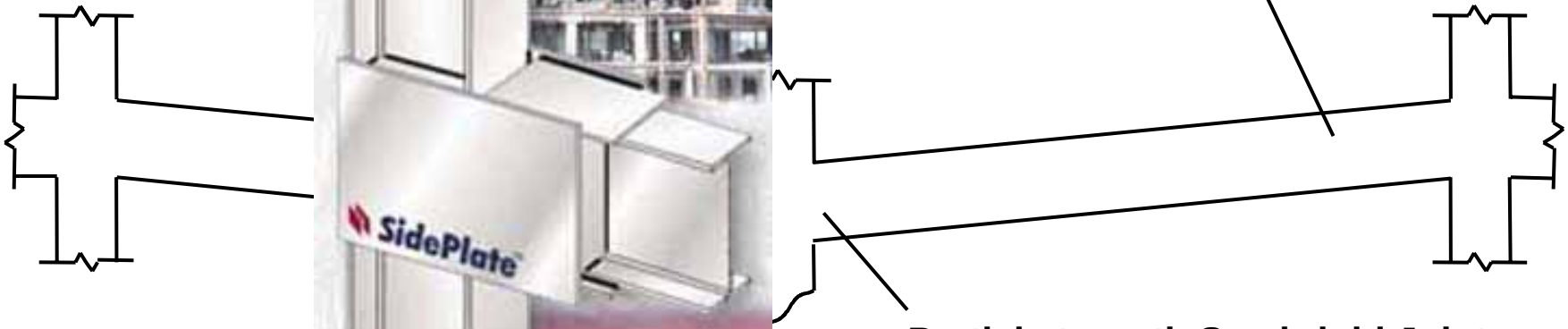
Stage 1 Fix beam to fin plate only

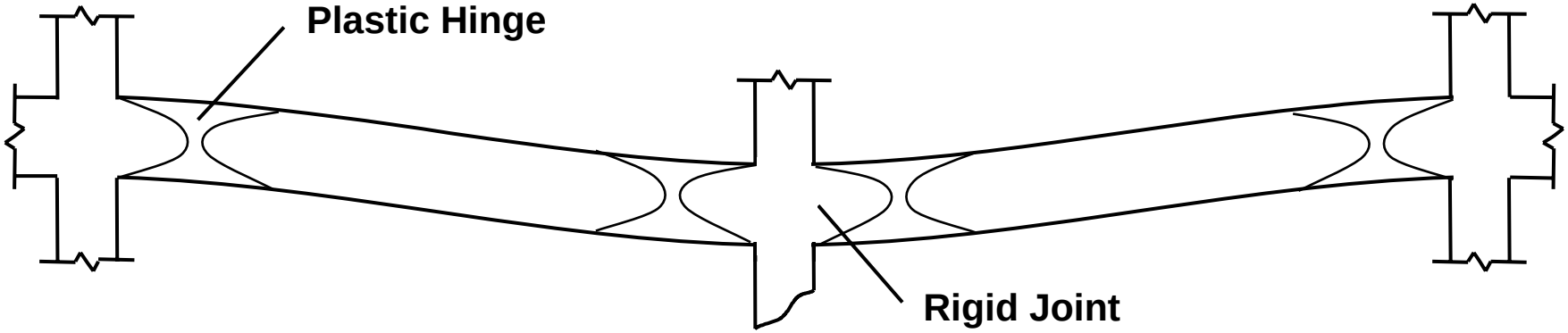
Stage 2 Link bars later and off critical path of construction programme

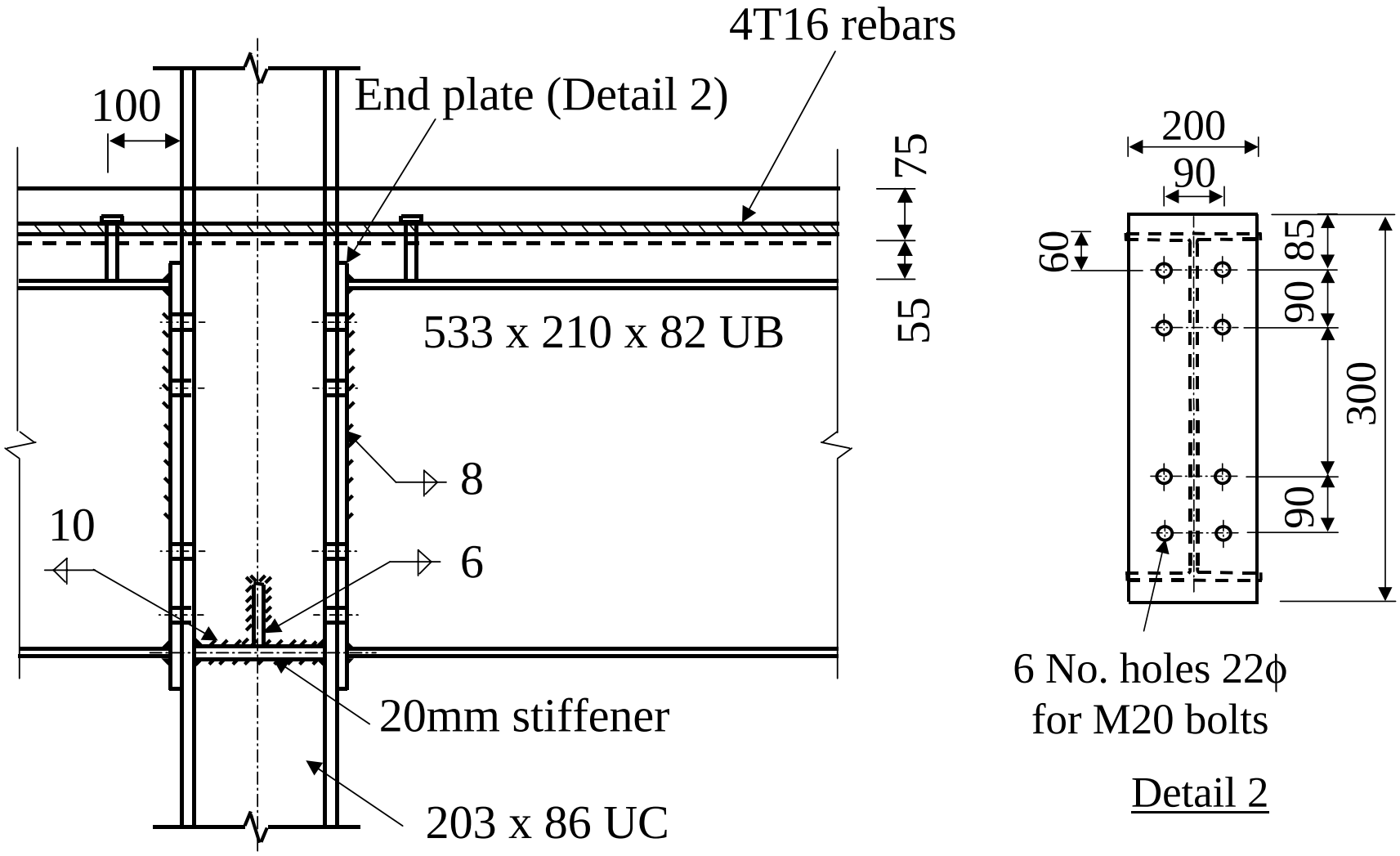
Available rotation capacity for industry standard semi-rigid composite connections limited to:

1.80^{o} for S355 bea

1.43^{o} for S275 bea



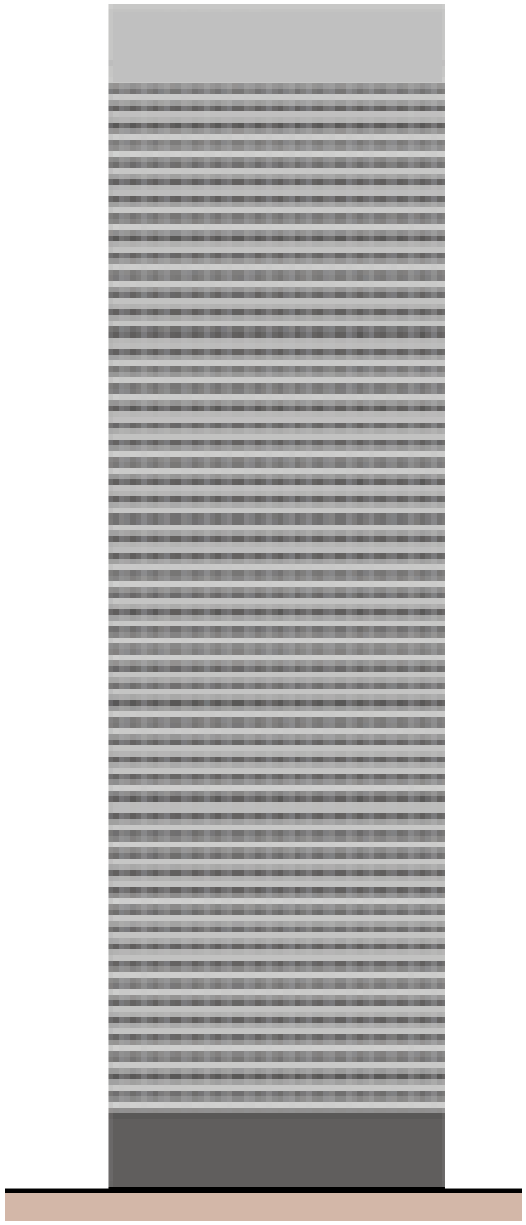


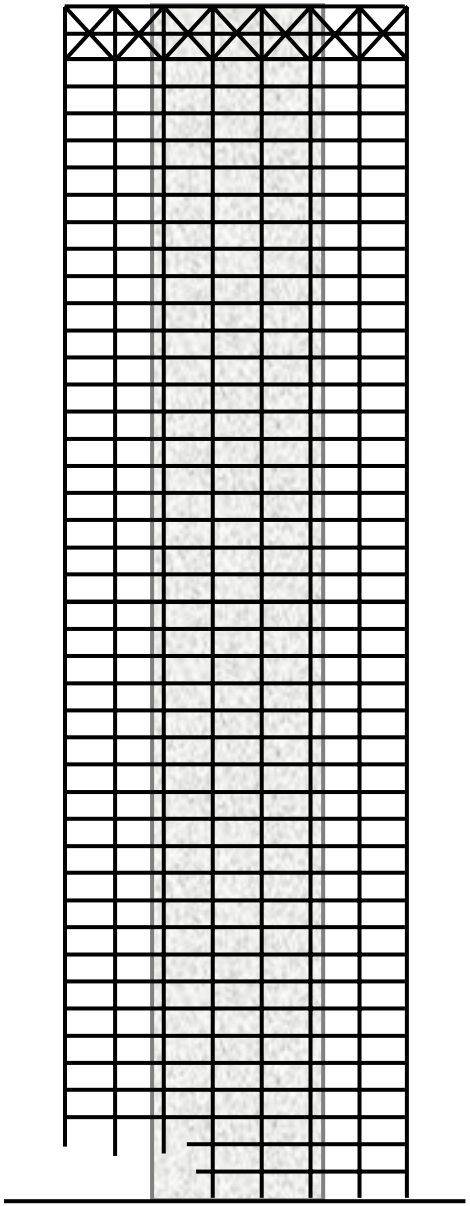

4T16 rebars
100
End plate (Detail 2)
75
55
533 x 210 x 82 UB
8
10
6
20mm stiffener
203 x 86 UC
200
90
60
85
90
300
90
6 No. holes 22ϕ
for M20 bolts

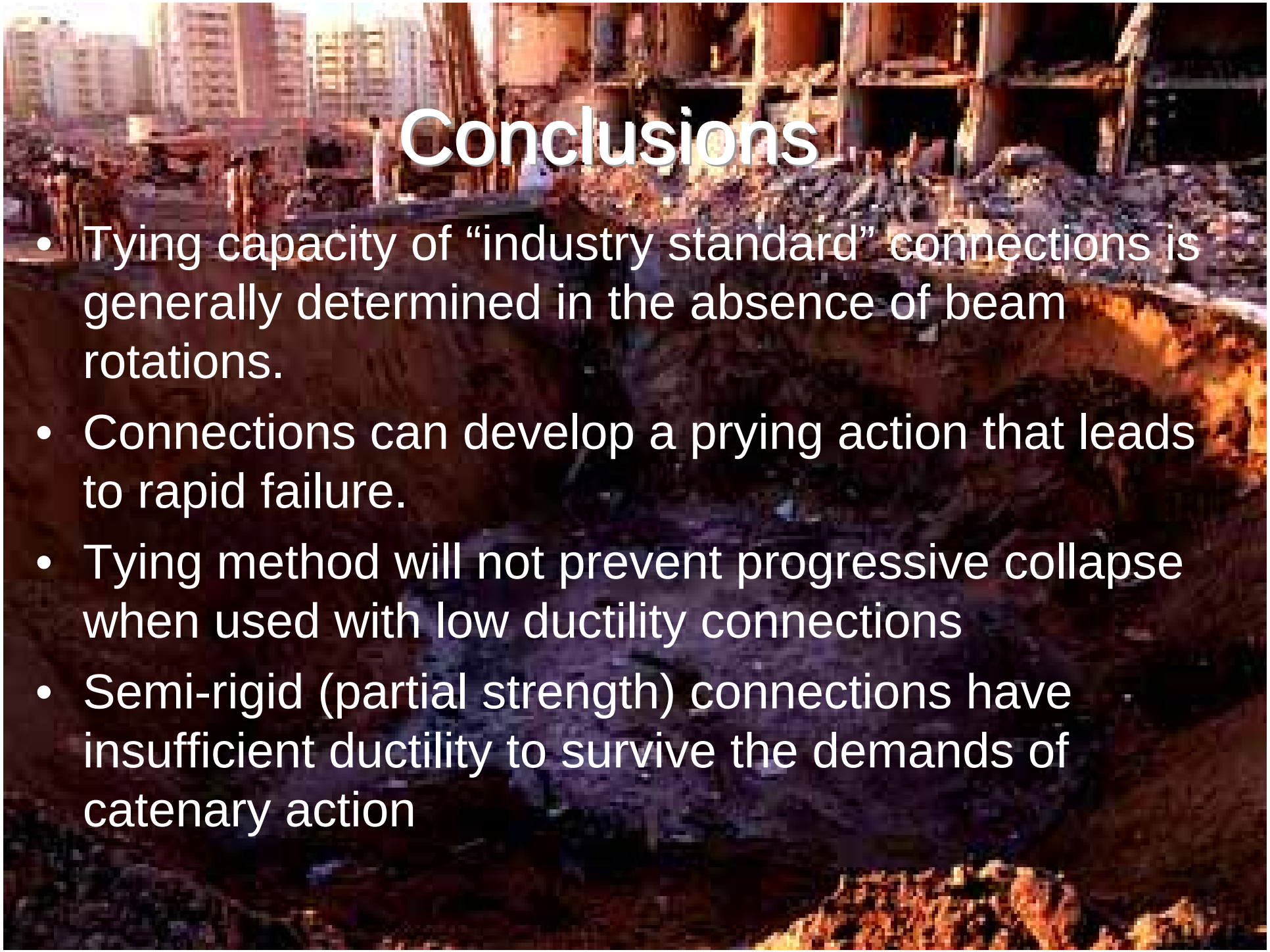
Detail 2


FRACTURED BOLT
DAMAGED BOLT
EXTERIOR COMPOSITE CONNECTION WEST SIDE
N


← DAMAGED BOLT
INTERIOR COMPOSITE
CONNECTION EAST SIDE
→N
Neutral
B3SEFBS 13.
FBN 17







Conclusions

- Tying capacity of “industry standard” connections is generally determined in the absence of beam rotations.
- Connections can develop a prying action that leads to rapid failure.
- Tying method will not prevent progressive collapse when used with low ductility connections
- Semi-rigid (partial strength) connections have insufficient ductility to survive the demands of catenary action