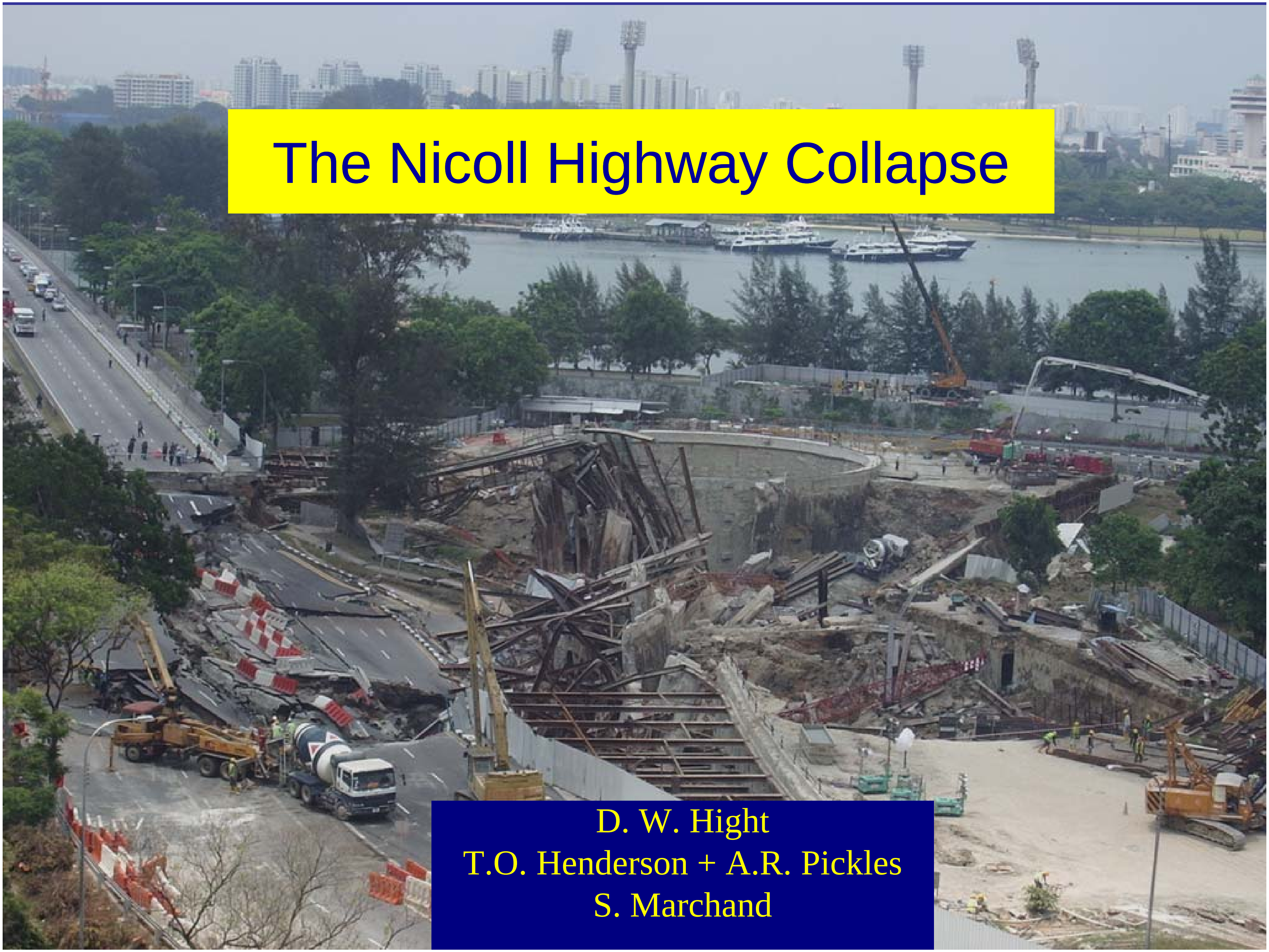
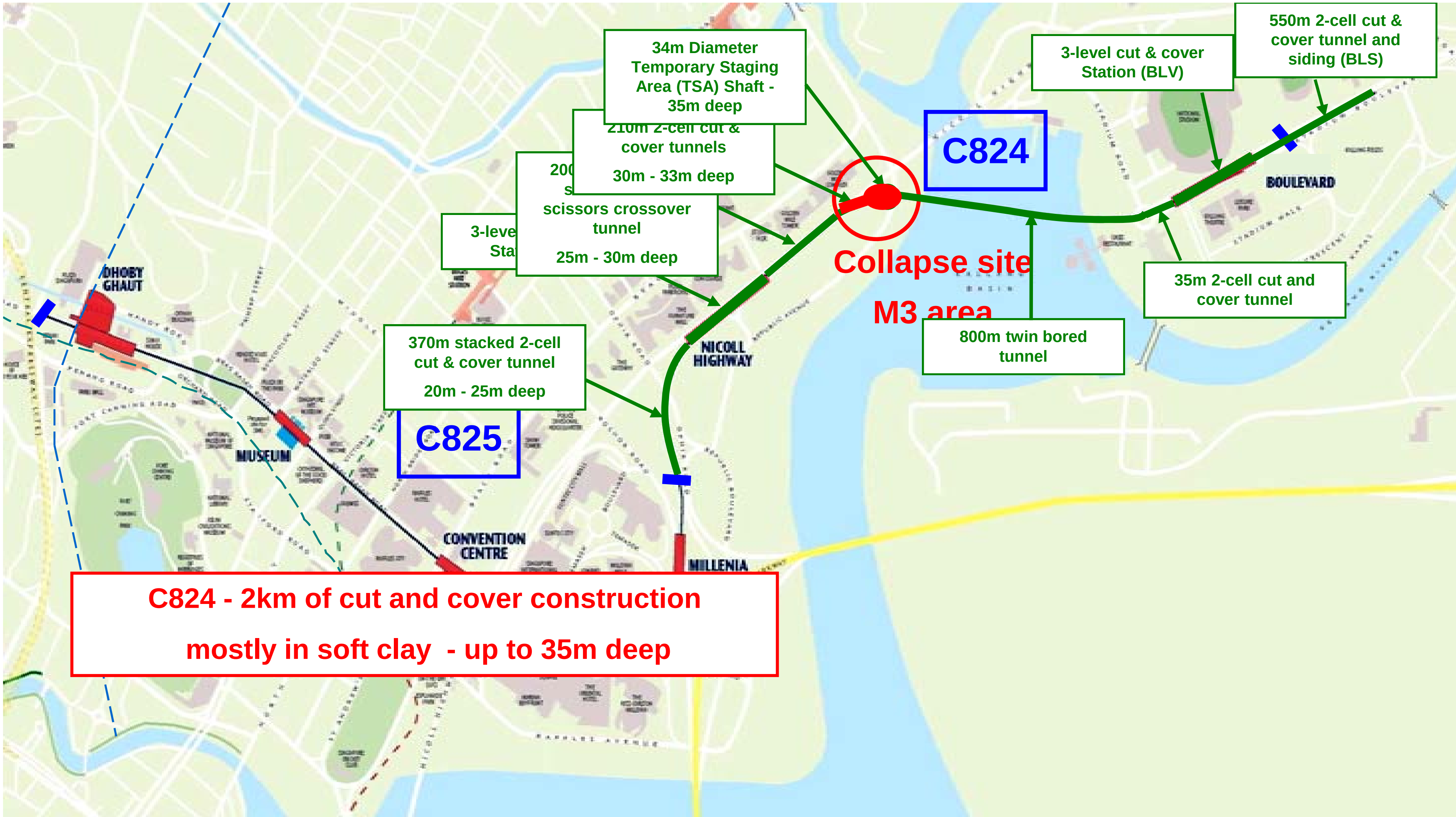
The Nicoll Highway Collapse



D. W. Hight
T.O. Henderson + A.R. Pickles
S. Marchand

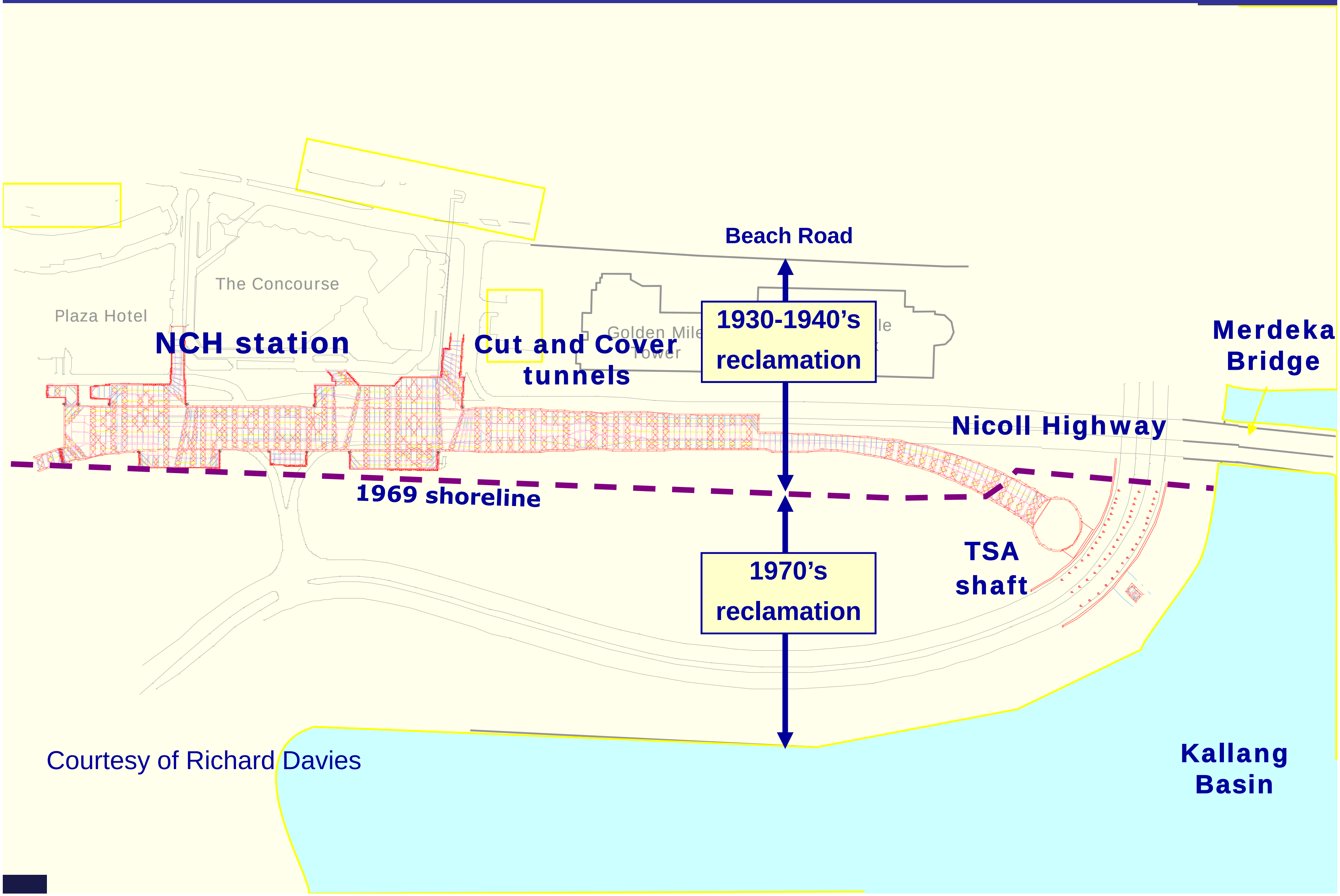
Content

- Background and construction sequence
- Observations up to the point of collapse, including monitoring
- The collapse
- Post-collapse investigations
- Design errors
- Jet grout layers
- Ground conditions and the buried valley
- The need for a trigger
- Back analyses of the collapse
- Relative vertical displacement and forced sway
- The bored piles
- Collapse mechanism and its trigger
- Lessons to be learnt

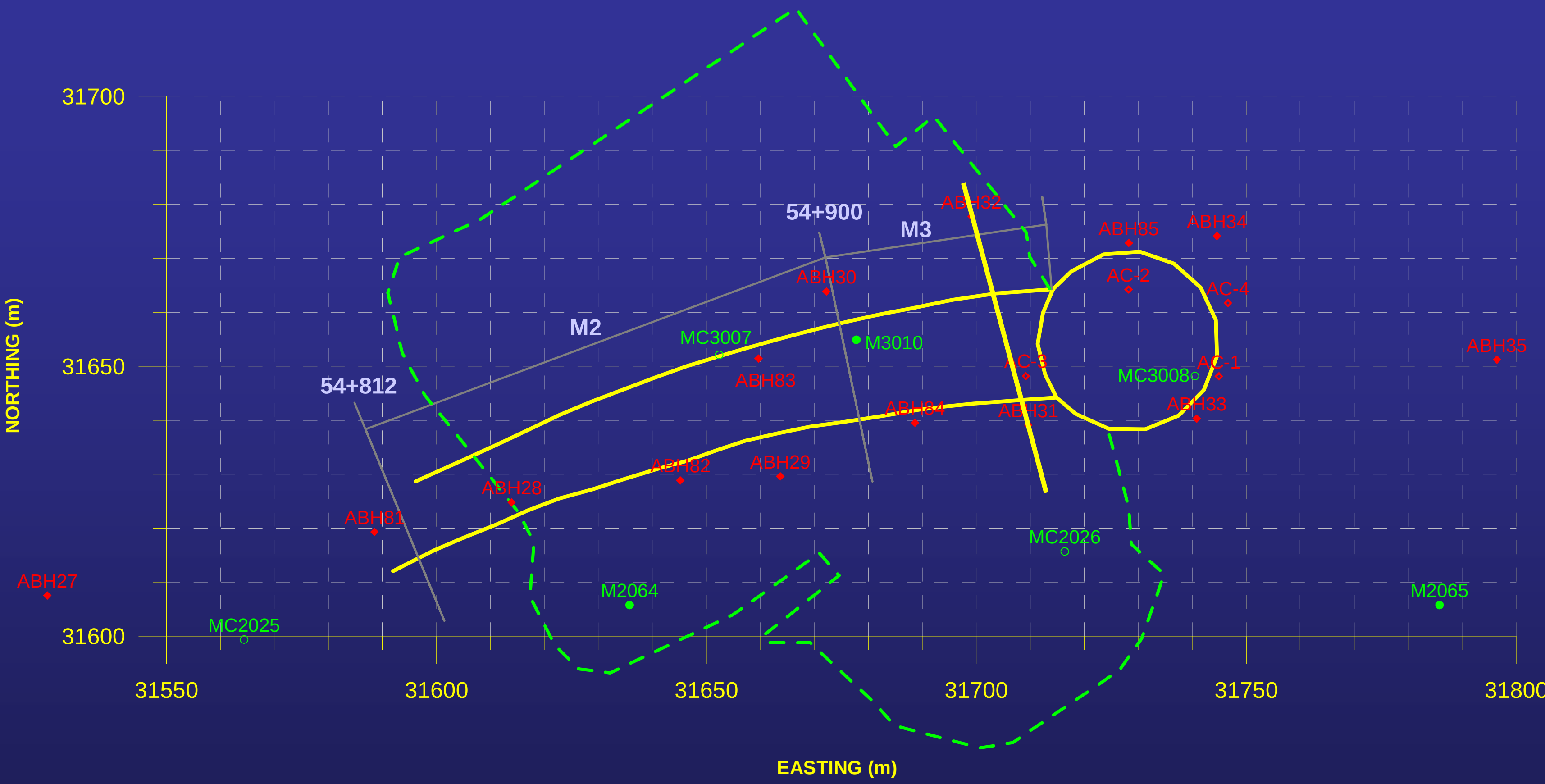


Circle Line Stage 1

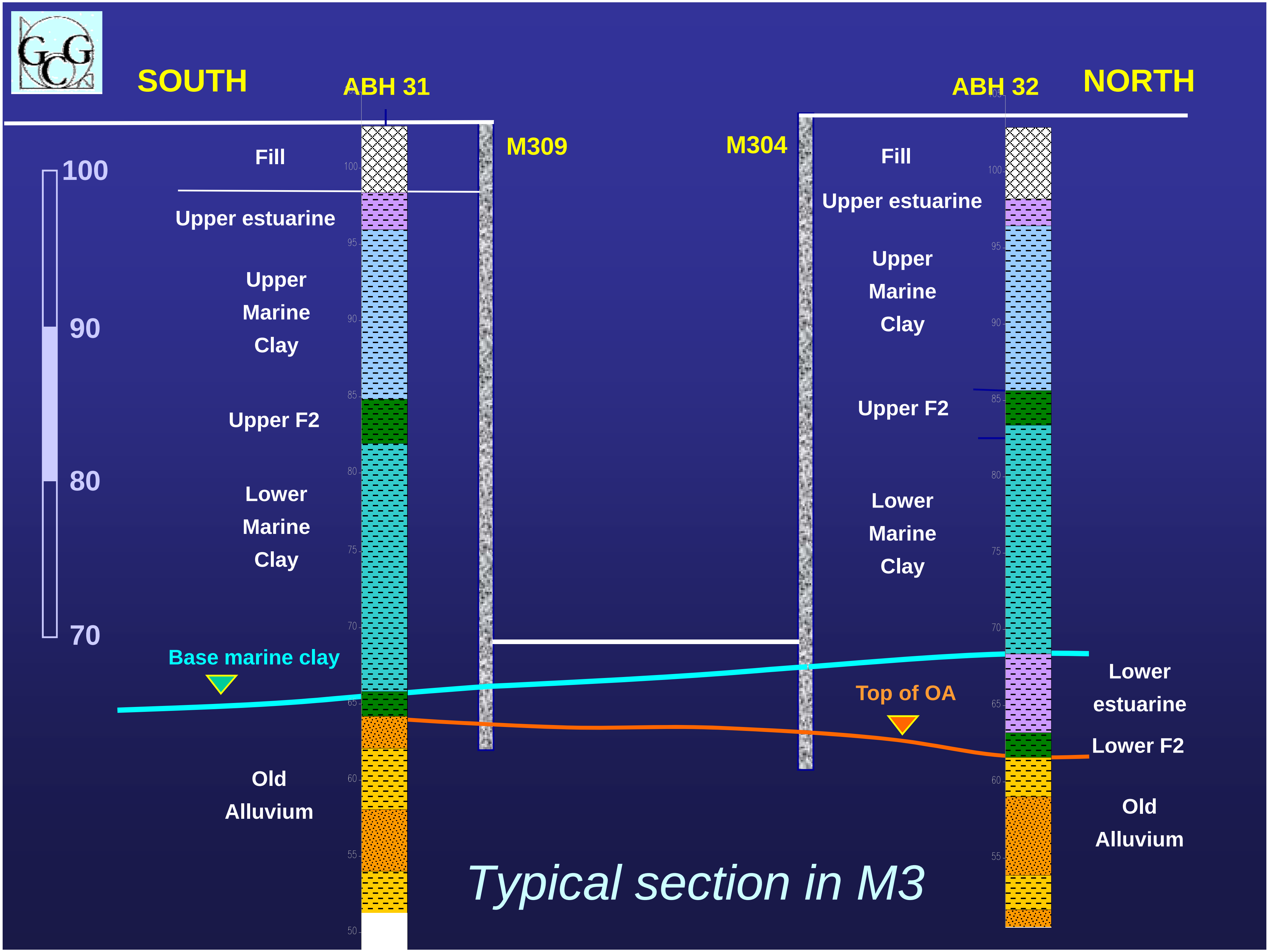
Reclamation dates



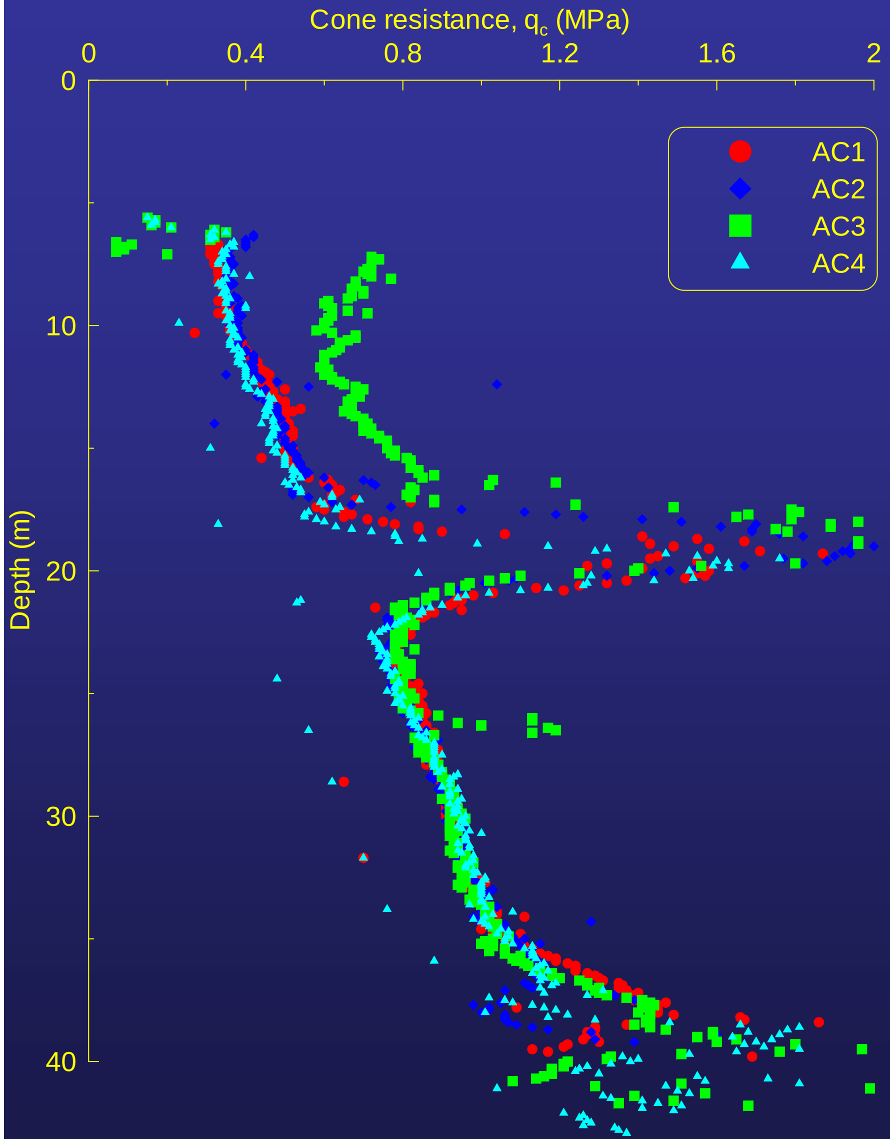
Courtesy of Richard Davies

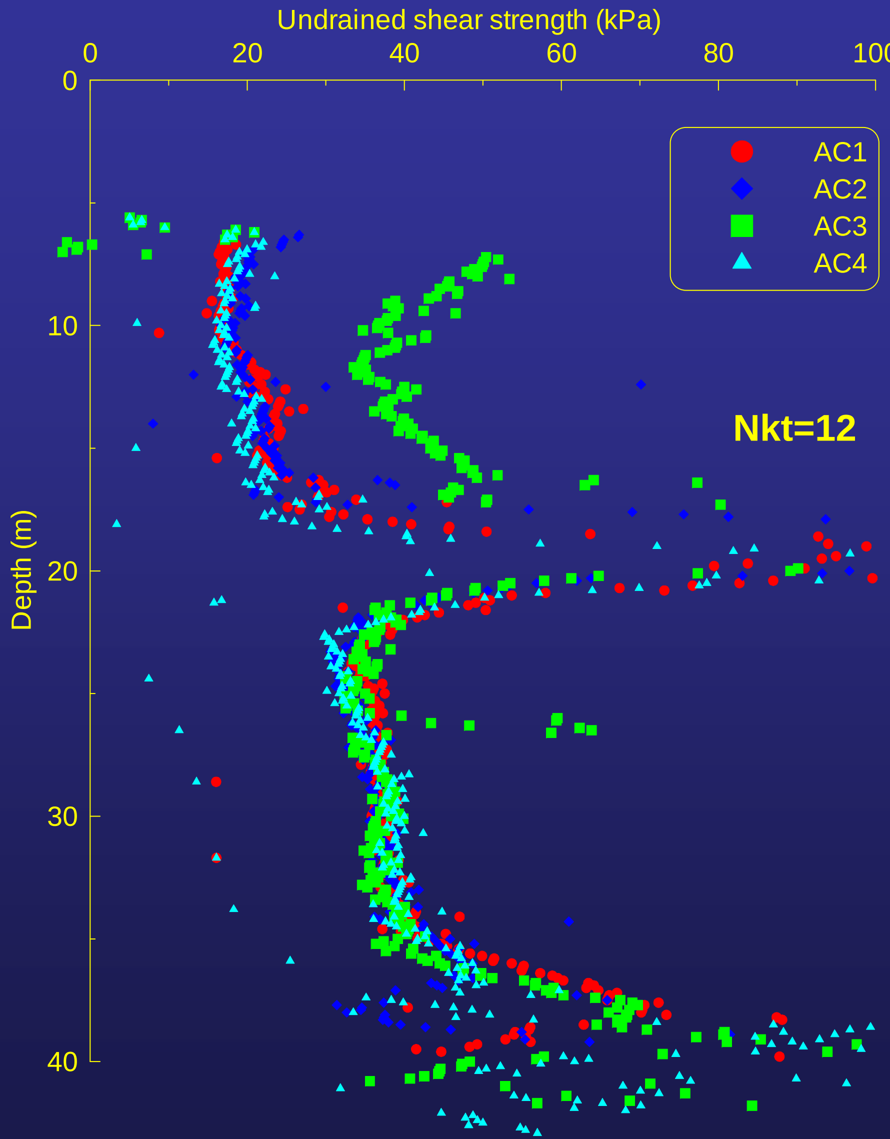


Pre- and post-tender site investigation



SOUTH
ABH 31
M309
M304
ABH 32
NORTH
100
90
80
70
Fill
Upper estuarine
Upper Marine Clay
Upper F2
Lower Marine Clay
Base marine clay
Old Alluvium
Top of OA
Lower estuarine
Lower F2
Typical section in M3





Piezocone data for M3 area

Construction details and sequence

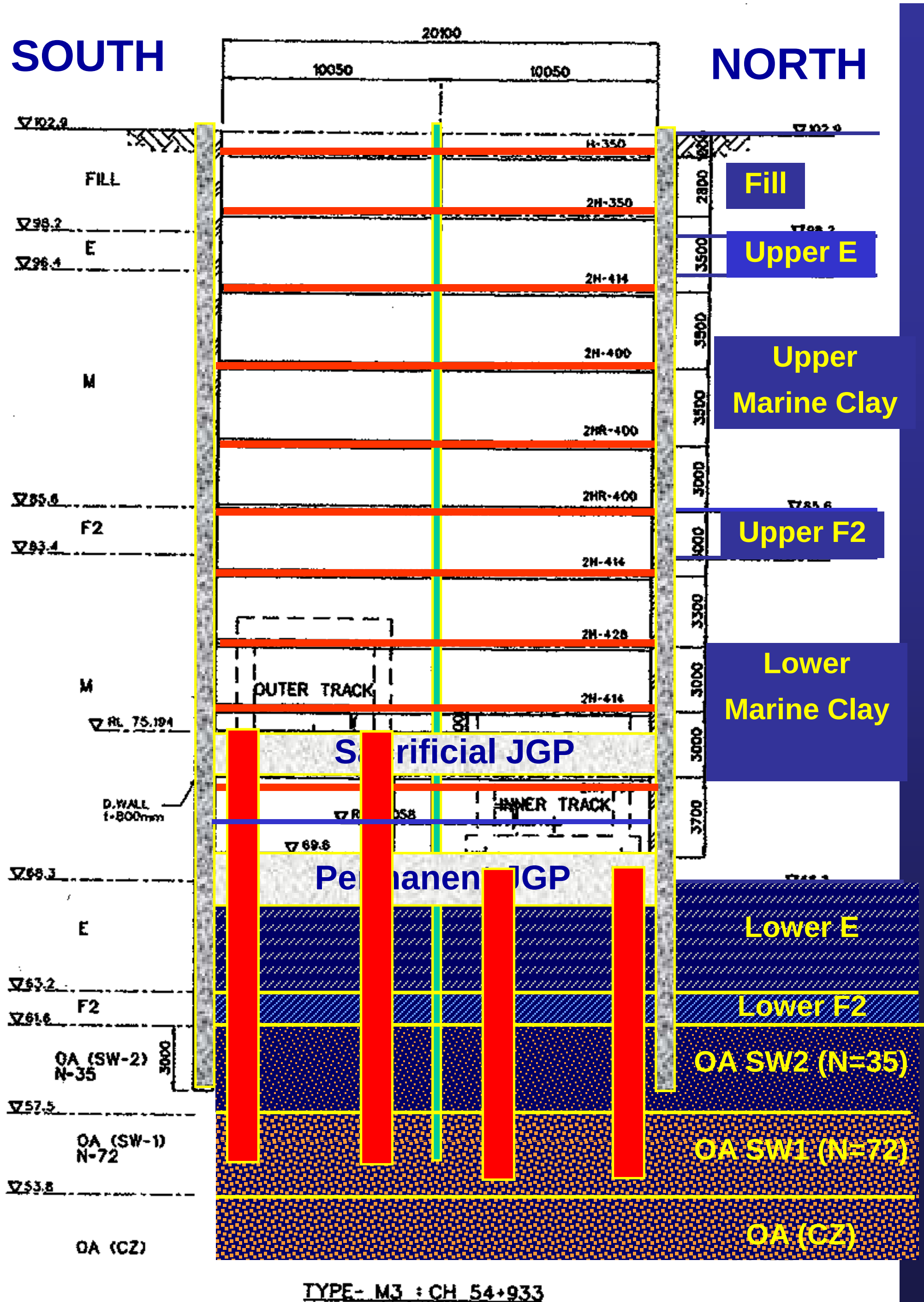
Formation level approx 33m bgl

Temporary diaphragm walls,0.8m

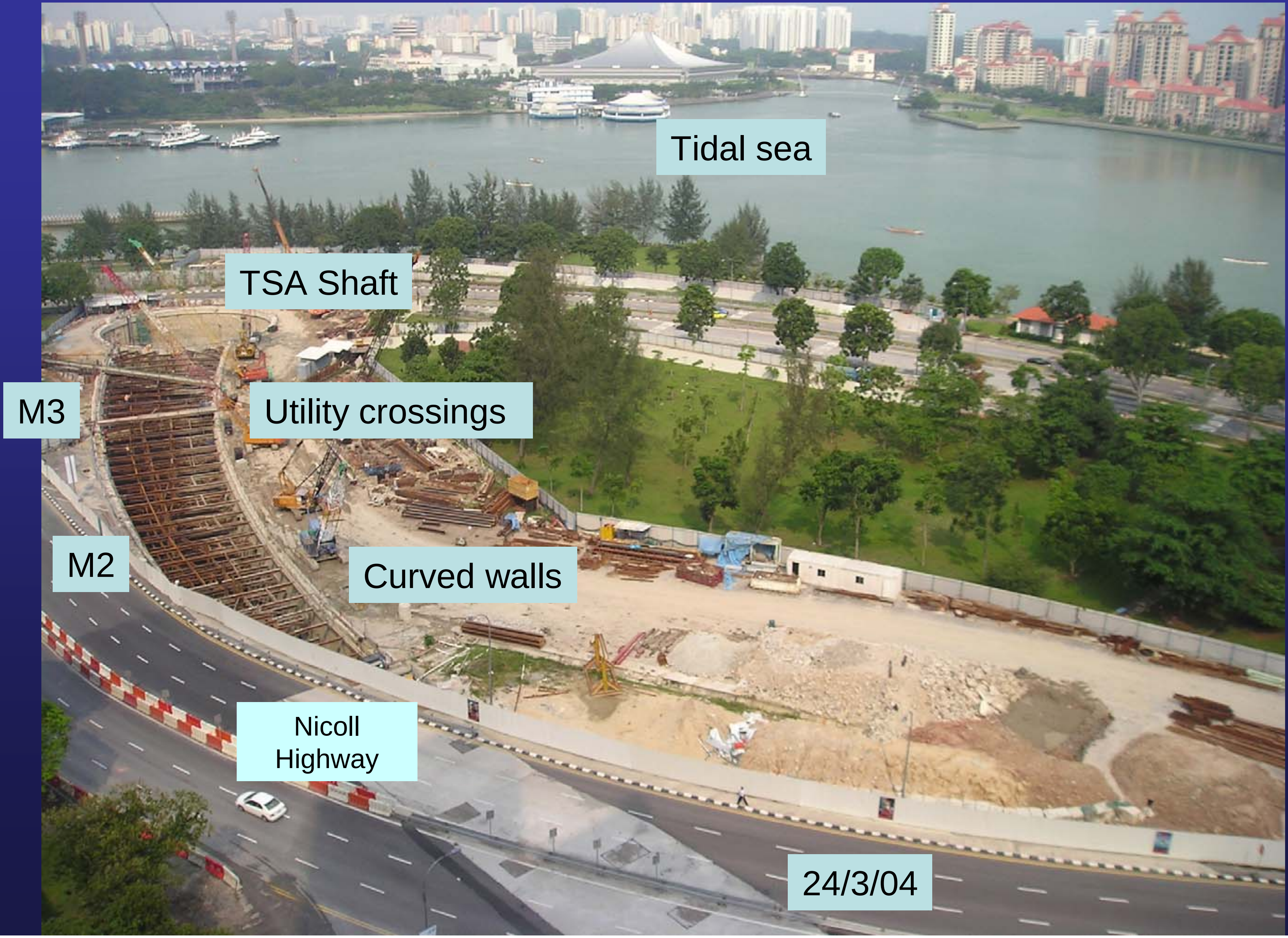
Driven kingpost

Permanent bored piles supporting rail boxes

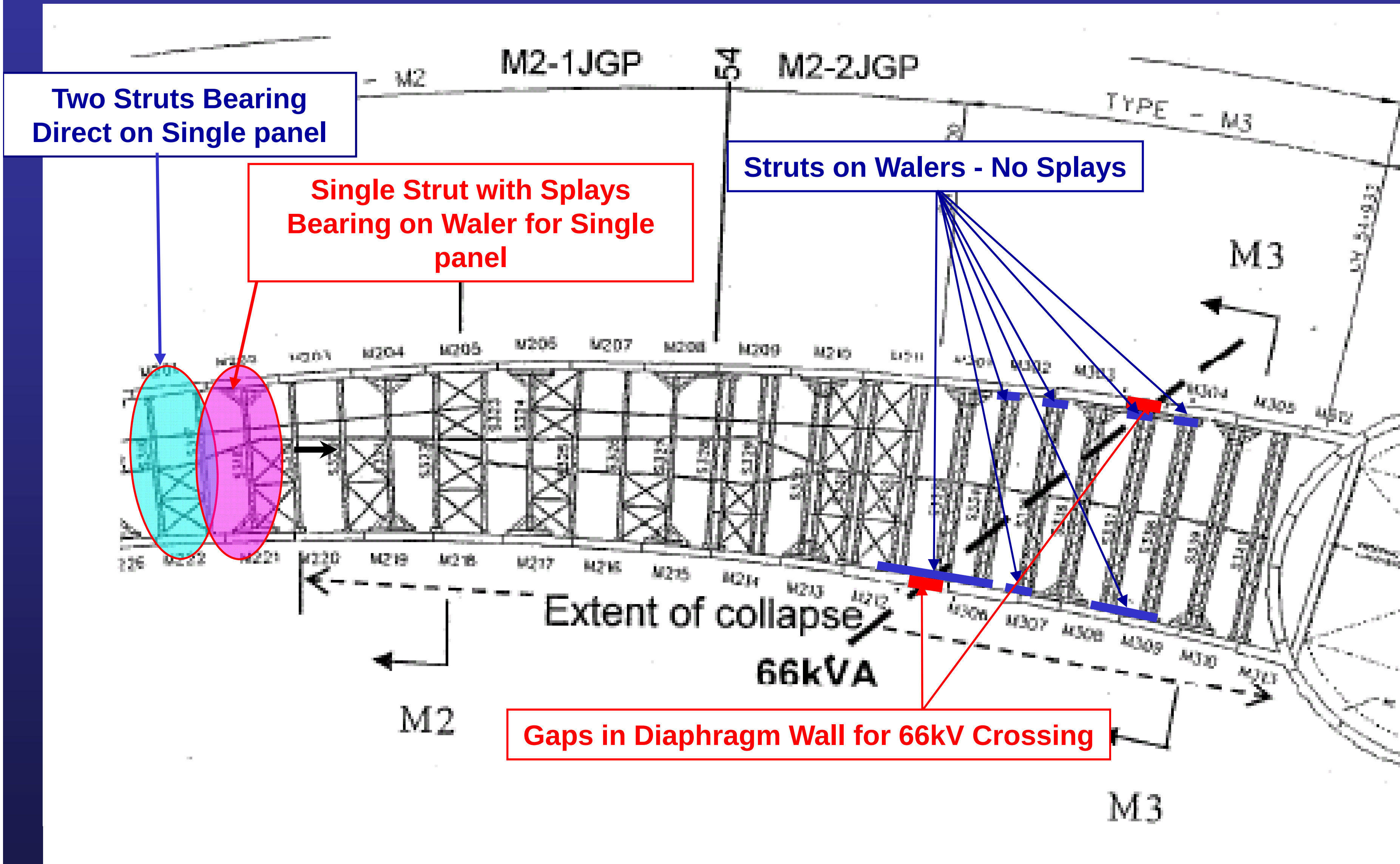
10 levels of steel struts at 3m +3.5m vertical centres



General Excavation sequence for M3 – up to level for removal of sacrificial JGP and installation of 10th level strut

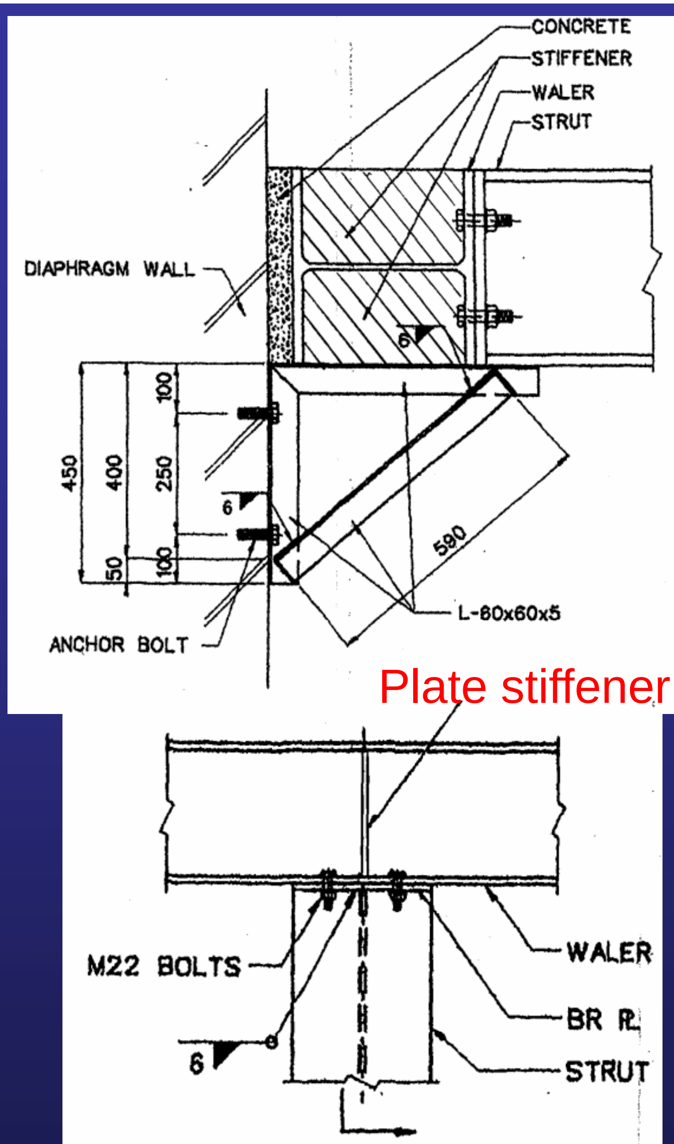

Tidal sea
TSA Shaft
M3
Utility crossings
M2
Curved walls
Nicoll
Highway
24/3/04

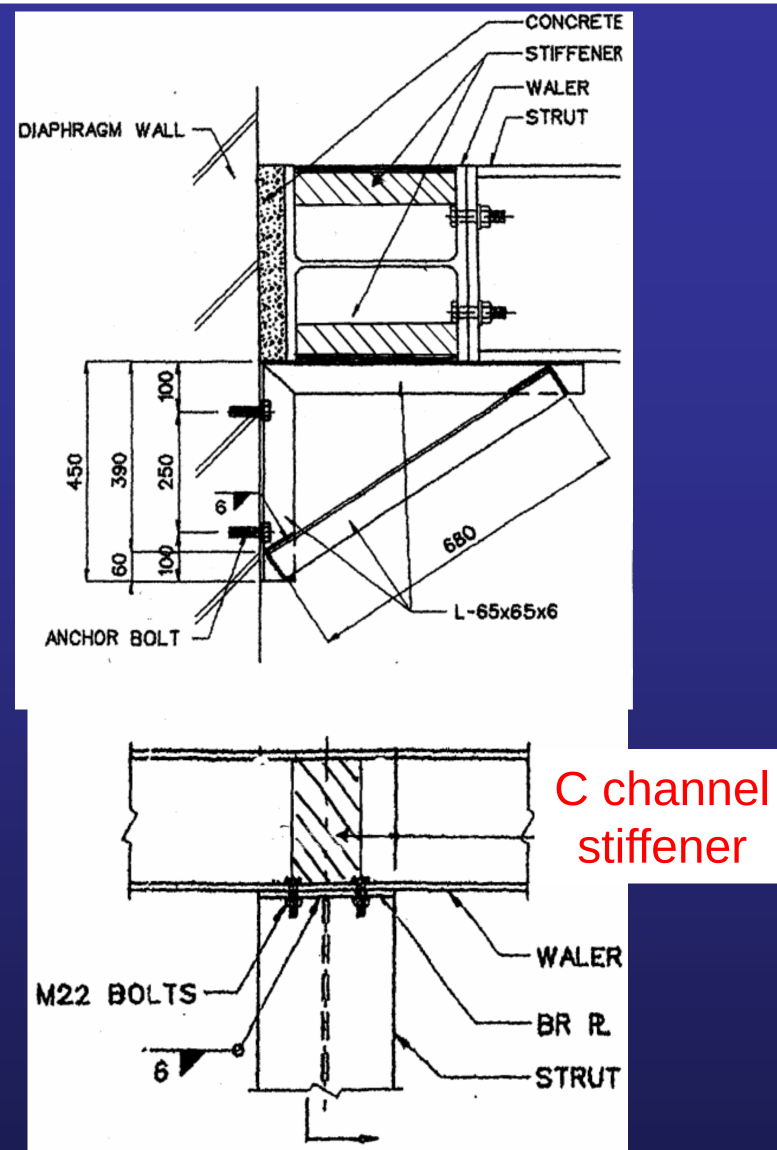

Support beam
Shaft
Kingpost
M3
No waler
C channel stiffener
Non-splayed strut
Jacking point
Walers
M2
Splayed strut


Two Struts Bearing Direct on Single panel
Single Strut with Splays Bearing on Waler for Single panel
Struts on Walers - No Splays
Gaps in Diaphragm Wall for 66kV Crossing
M2-1JGP
54
M2-2JGP
TYPE - M3
M3
M2
Extent of collapse
66kVA
M3

South side 13 March 04

Events and observations prior to collapse

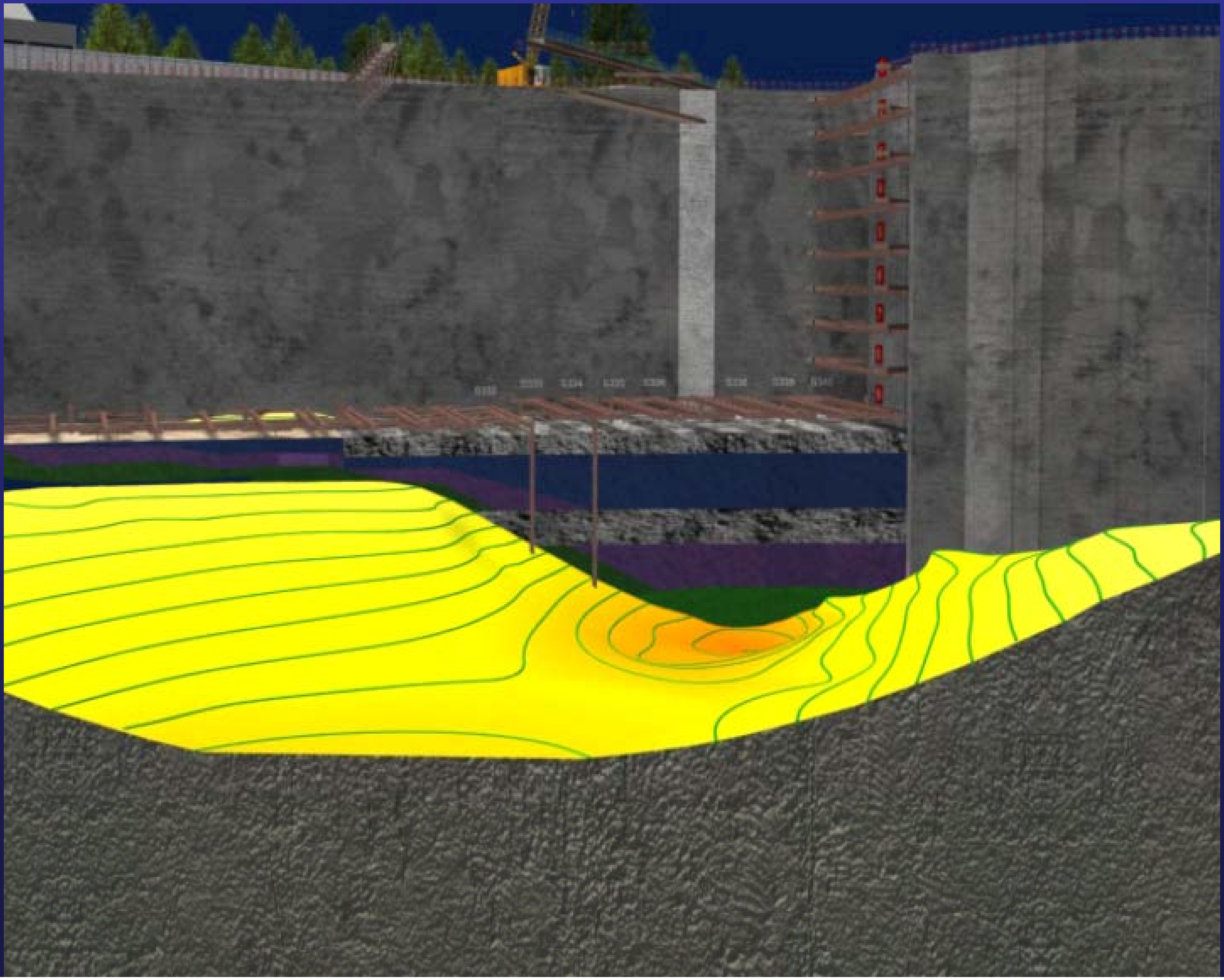




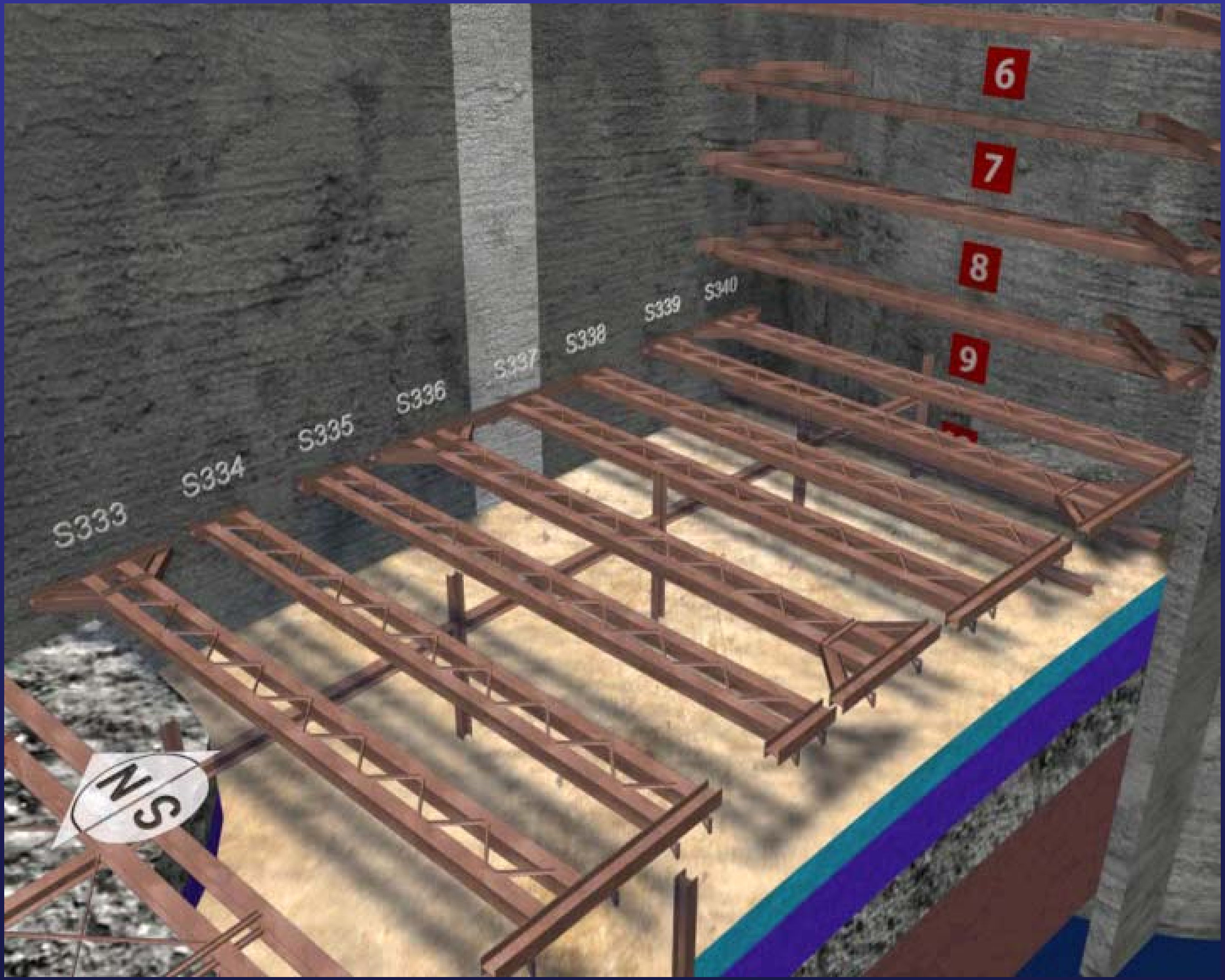
Replacement of plate stiffeners at strut-waler connection


Strut bearing directly on Dwall
C channel connection

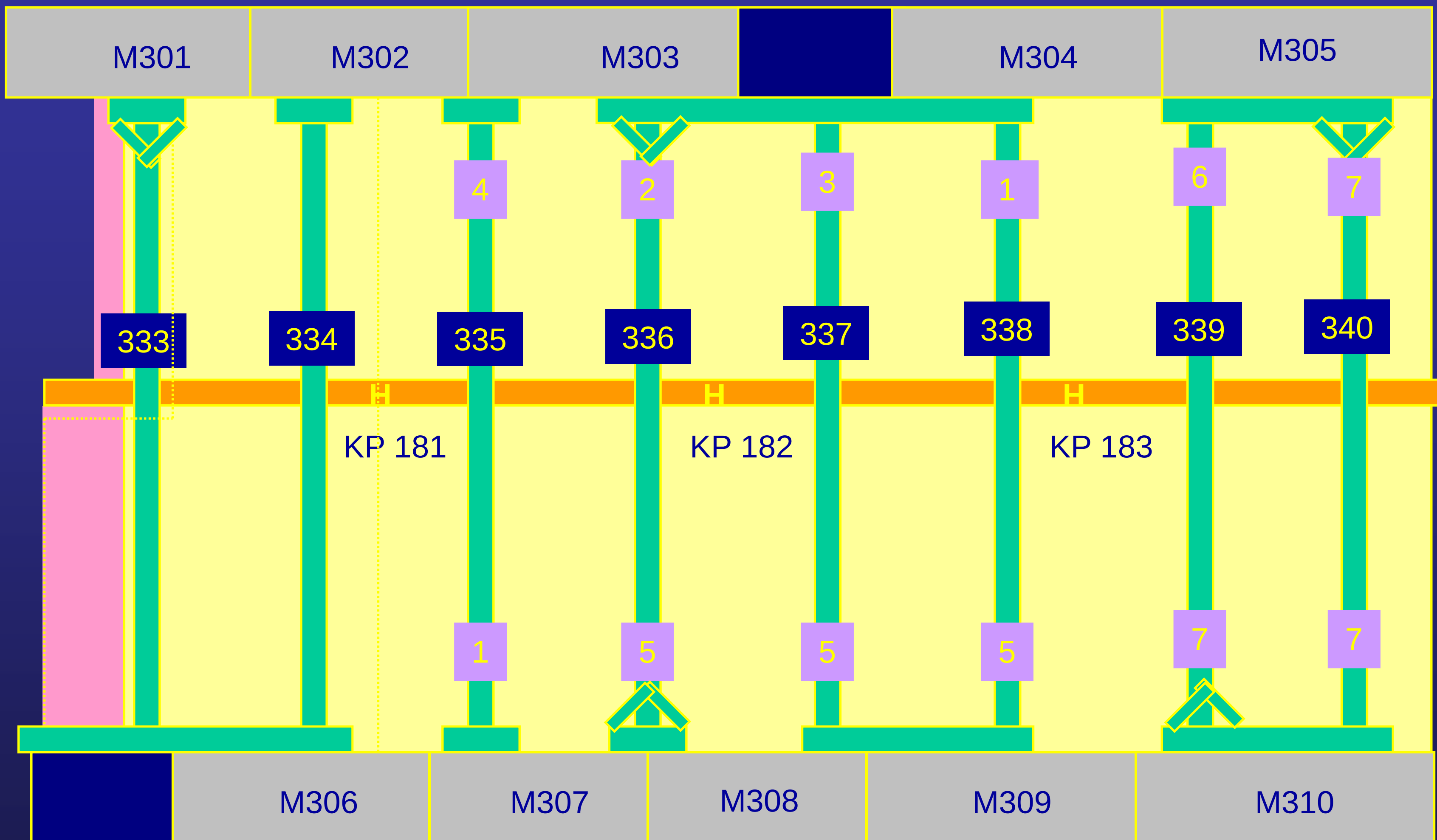
Excavation for the 10th level of struts, including removal of the sacrificial JGP



Observations on the morning of the collapse



North wall



1 Order in which distortion to waler noted

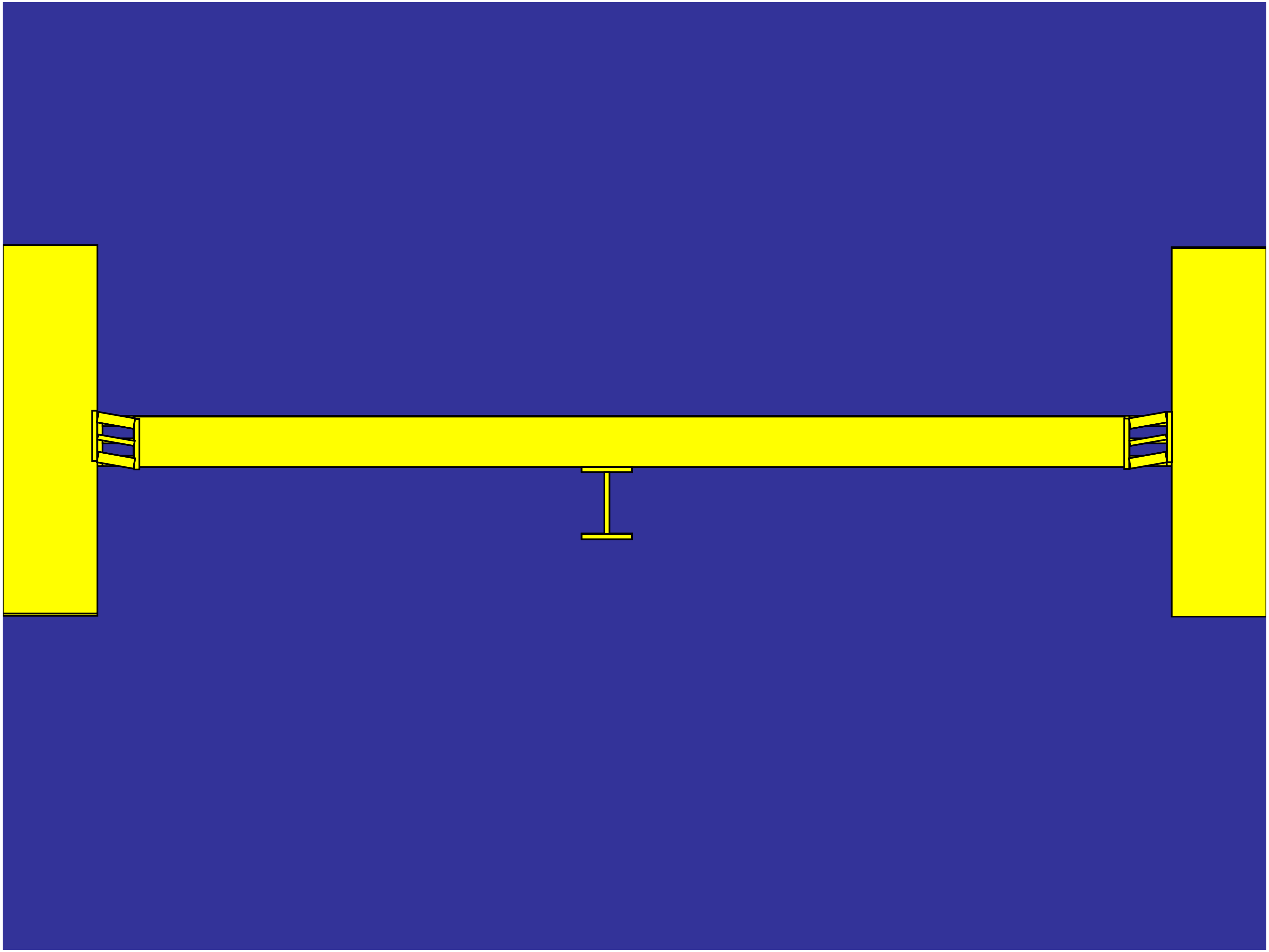
Excavation in progress

Excavation to 10th complete

Location of walers, splays and Dwall gaps

Strut 335-9 south wall

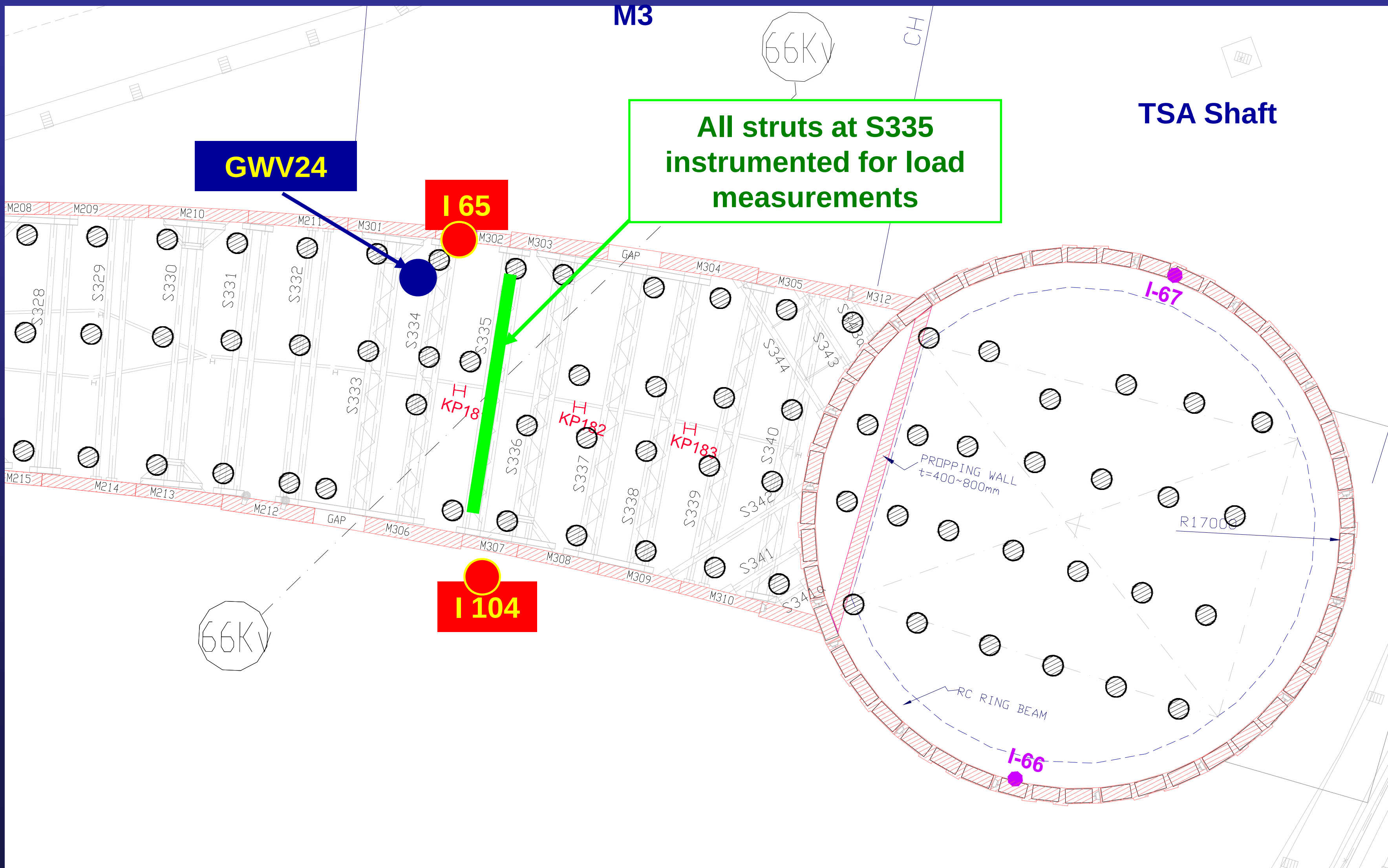
Strut 338-9 north wall

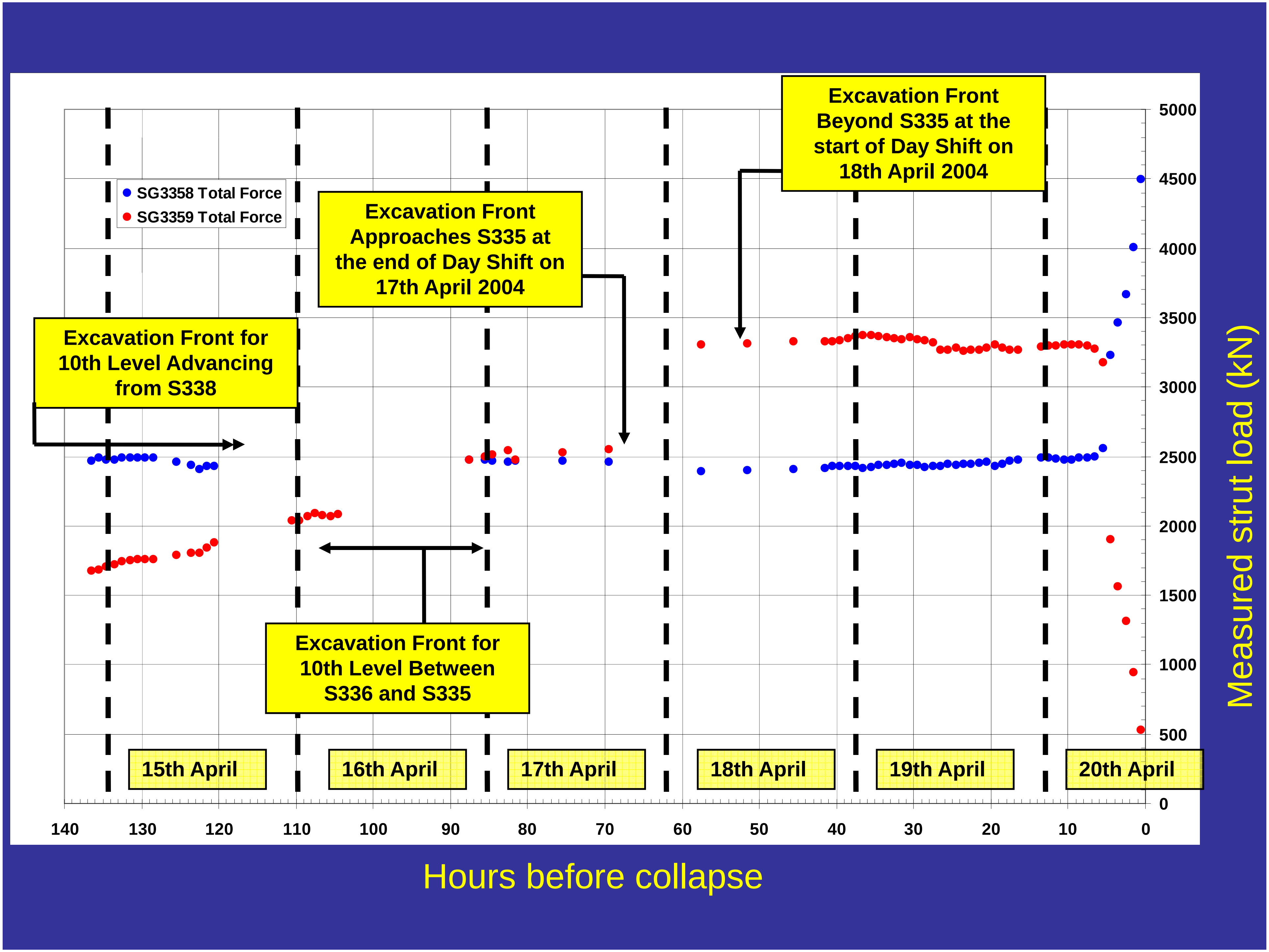


Instrumentation and results of monitoring

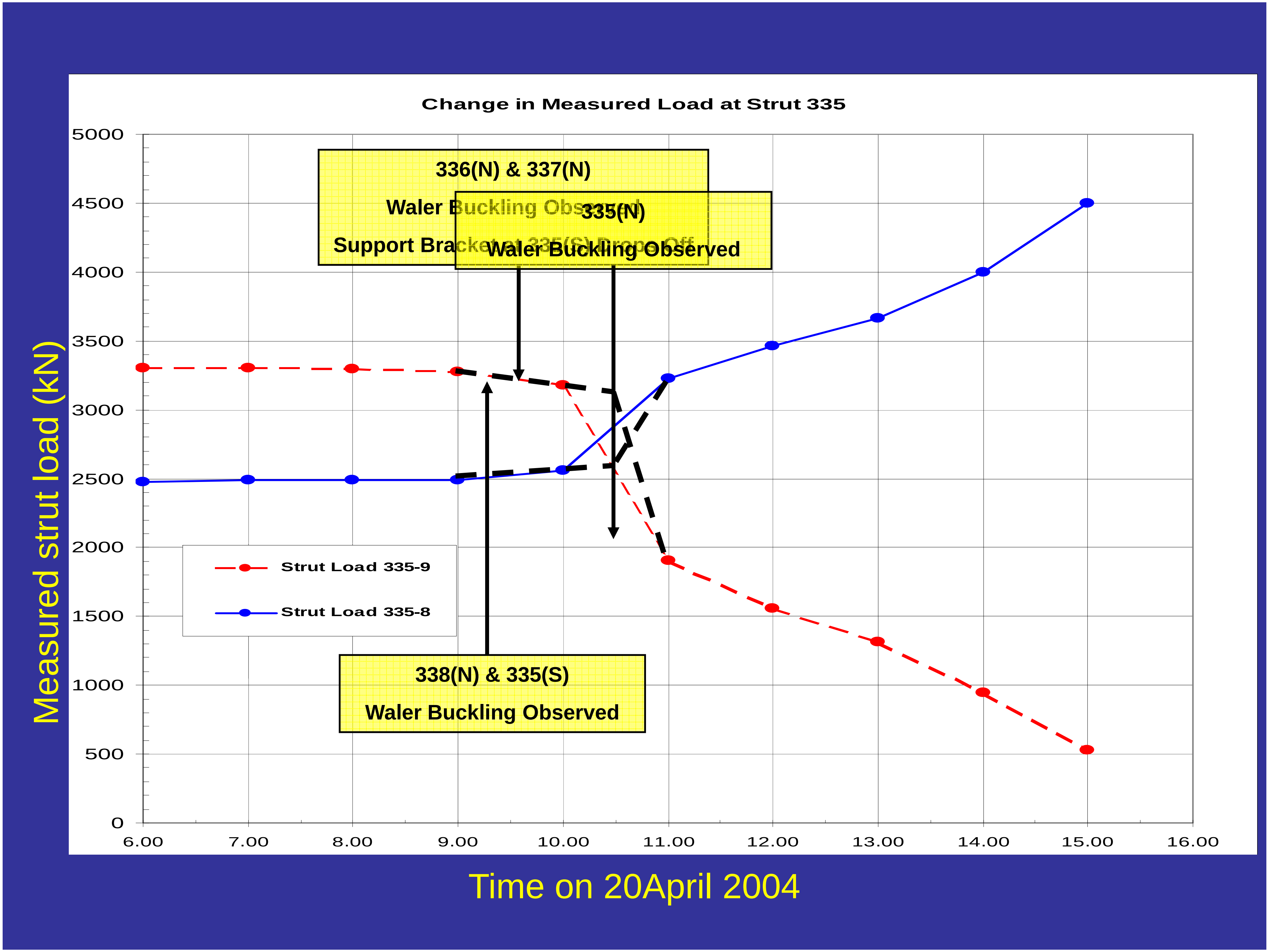
M2/M3 plan at 9th level

M2

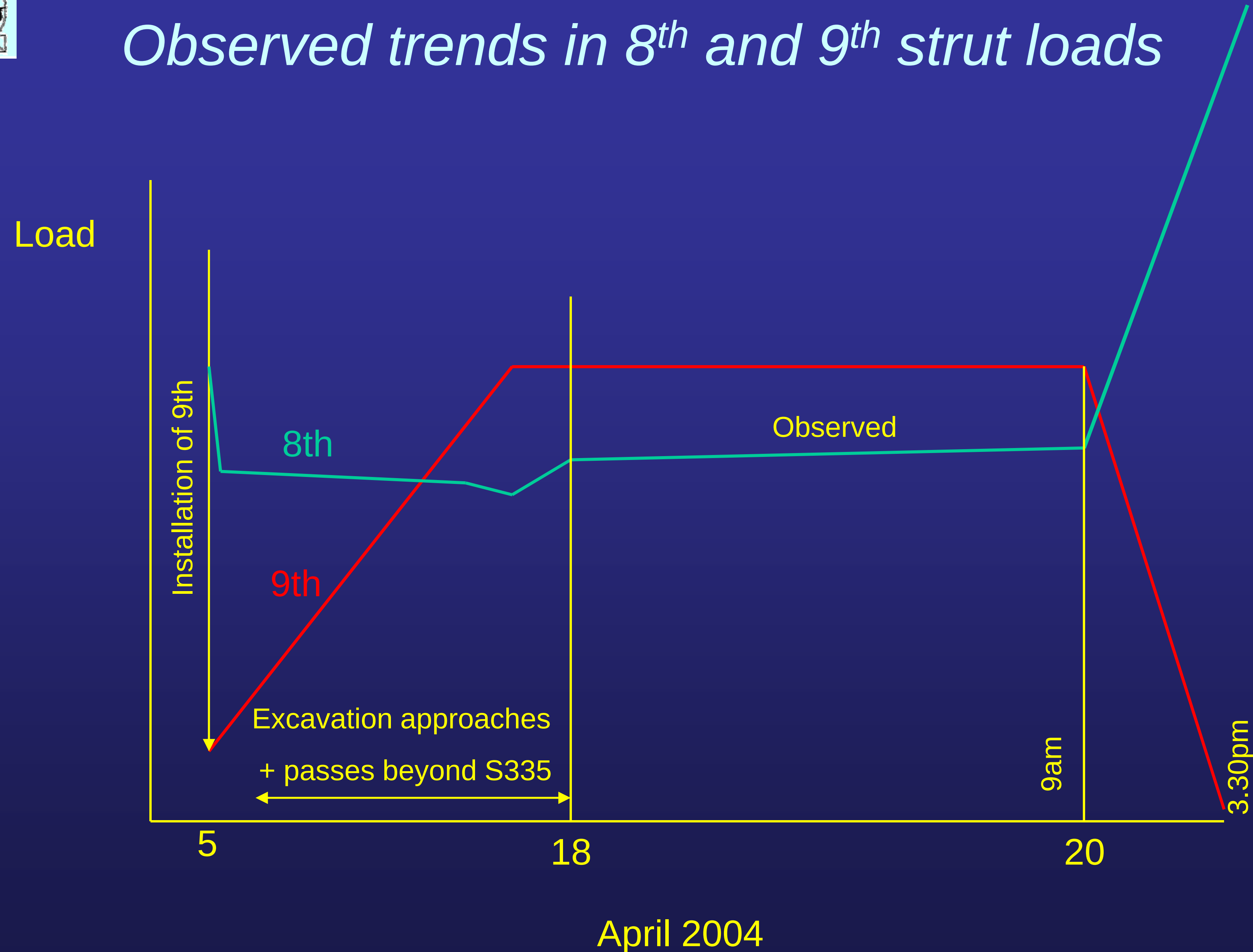




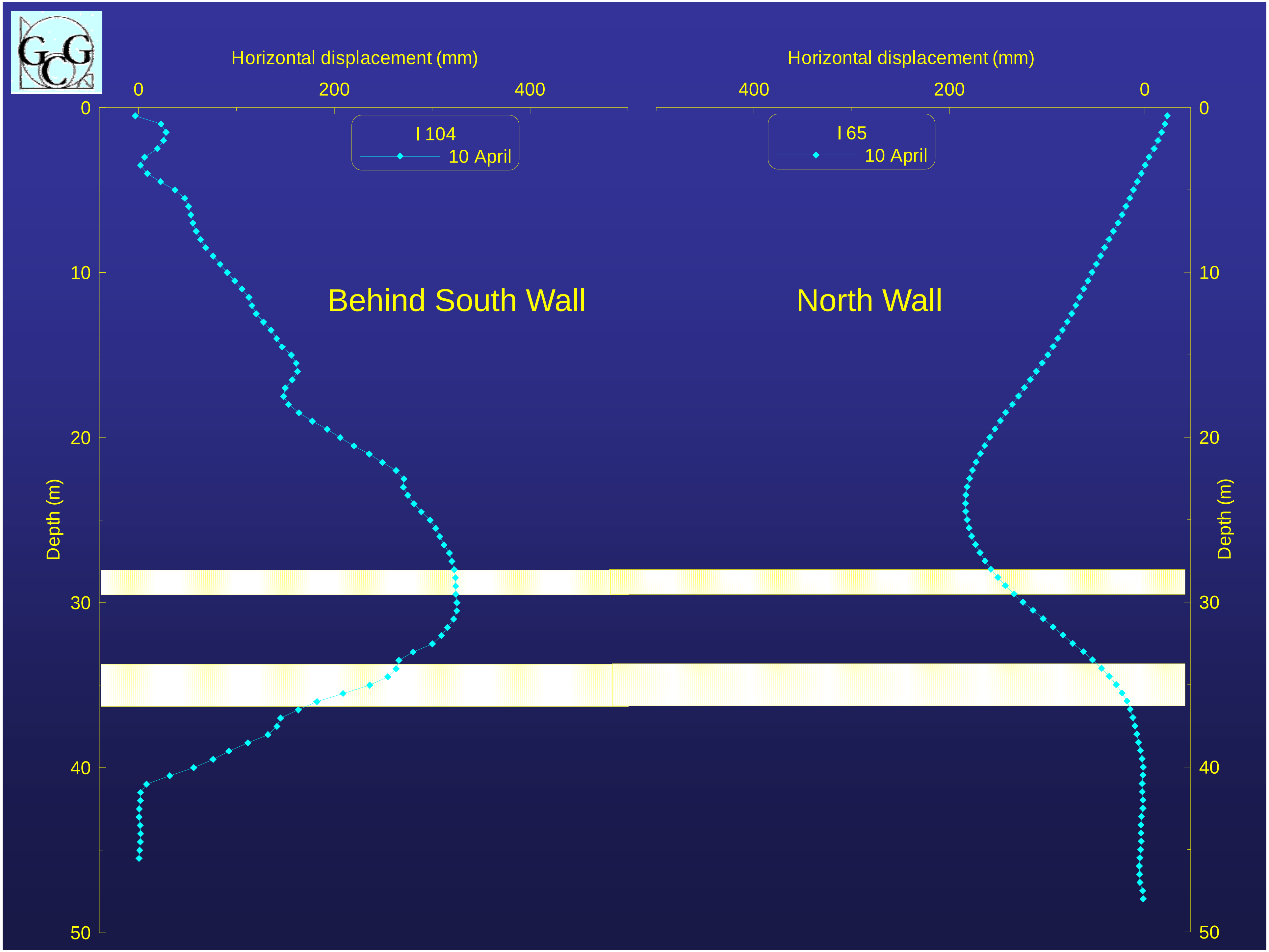
Excavation Front Beyond S335 at the start of Day Shift on 18th April 2004
Excavation Front Approaches S335 at the end of Day Shift on 17th April 2004
Excavation Front for 10th Level Advancing from S338
Excavation Front for 10th Level Between S336 and S335
SG3358 Total Force
SG3359 Total Force
15th April
16th April
17th April
18th April
19th April
20th April
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
Measured strut load (kN)
Hours before collapse


Change in Measured Load at Strut 335
336(N) & 337(N)
Waler Buckling Observed
Support Bracket at 335(S) Drops Off
335(N)
Waler Buckling Observed
338(N) & 335(S)
Waler Buckling Observed
Strut Load 335-9
Strut Load 335-8
Measured strut load (kN)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
6.00
7.00
8.00
9.00
10.00
11.00
12.00
13.00
14.00
15.00
16.00
Time on 20April 2004

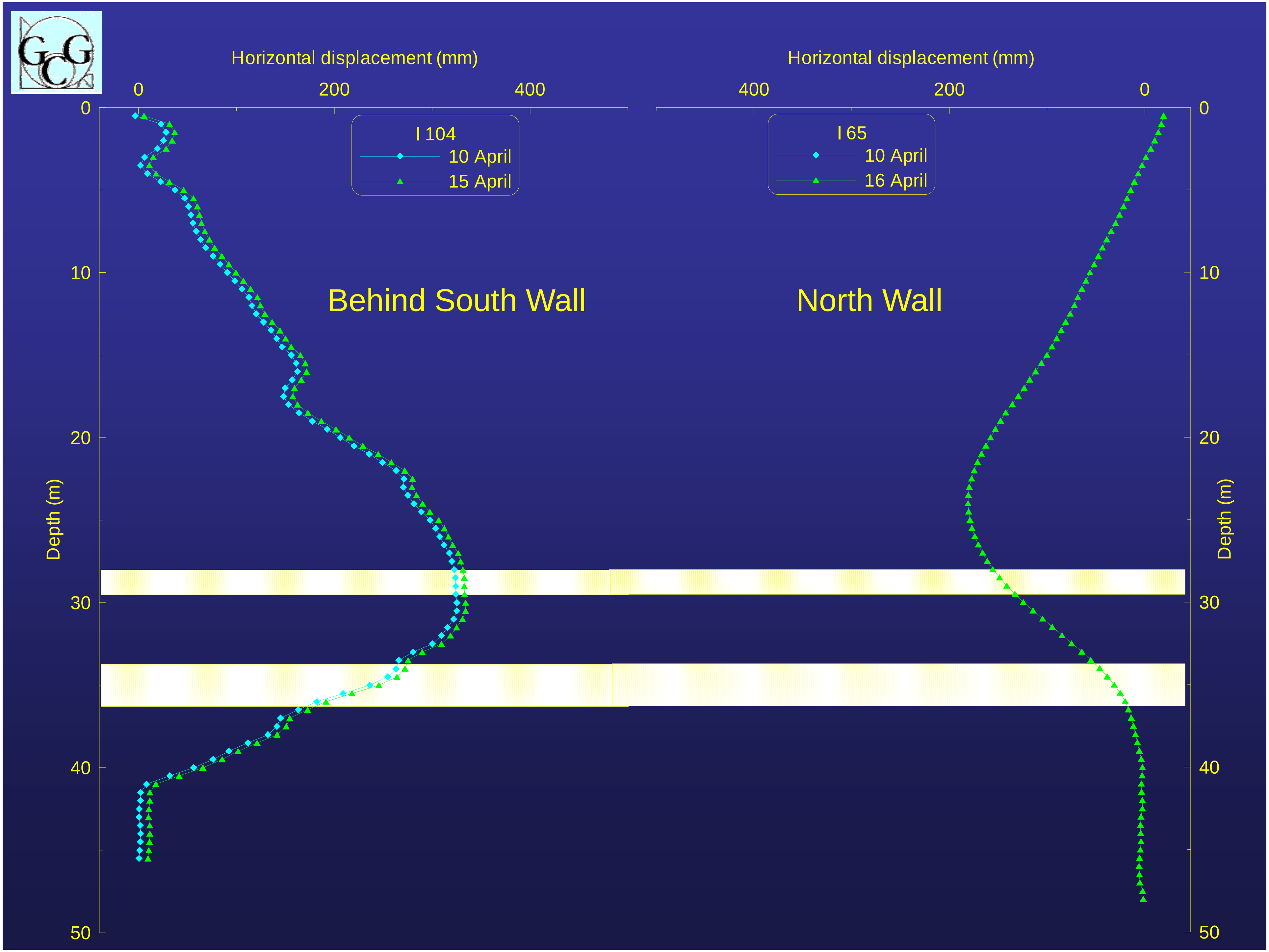
Observed trends in 8th and 9th strut loads



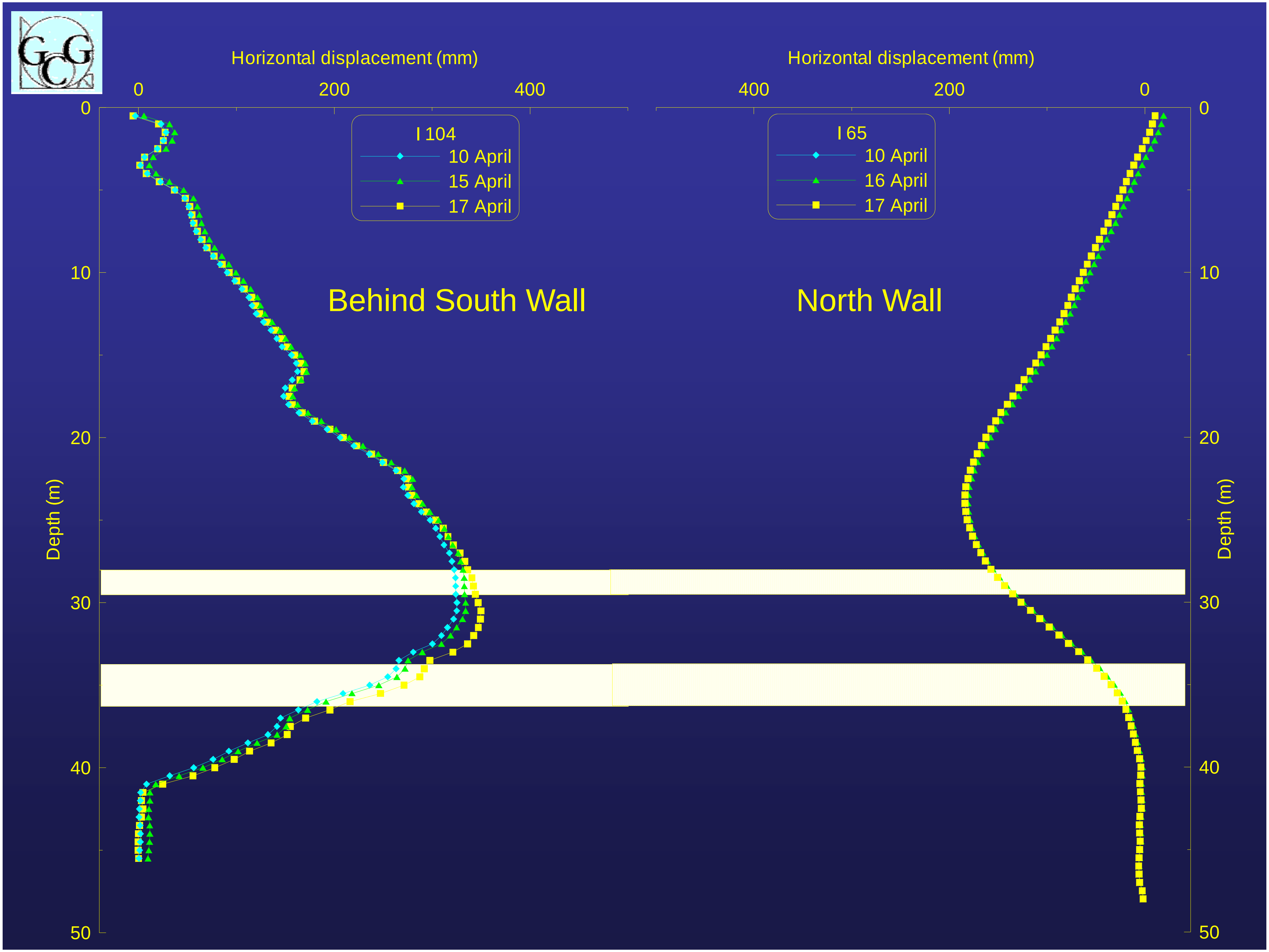
The trends were consistent with there being yielding of the 9th level strut-waler connection when the excavation passed beneath but with no further significant changes in load in either the 9th or 8th level struts until the collapse was initiated



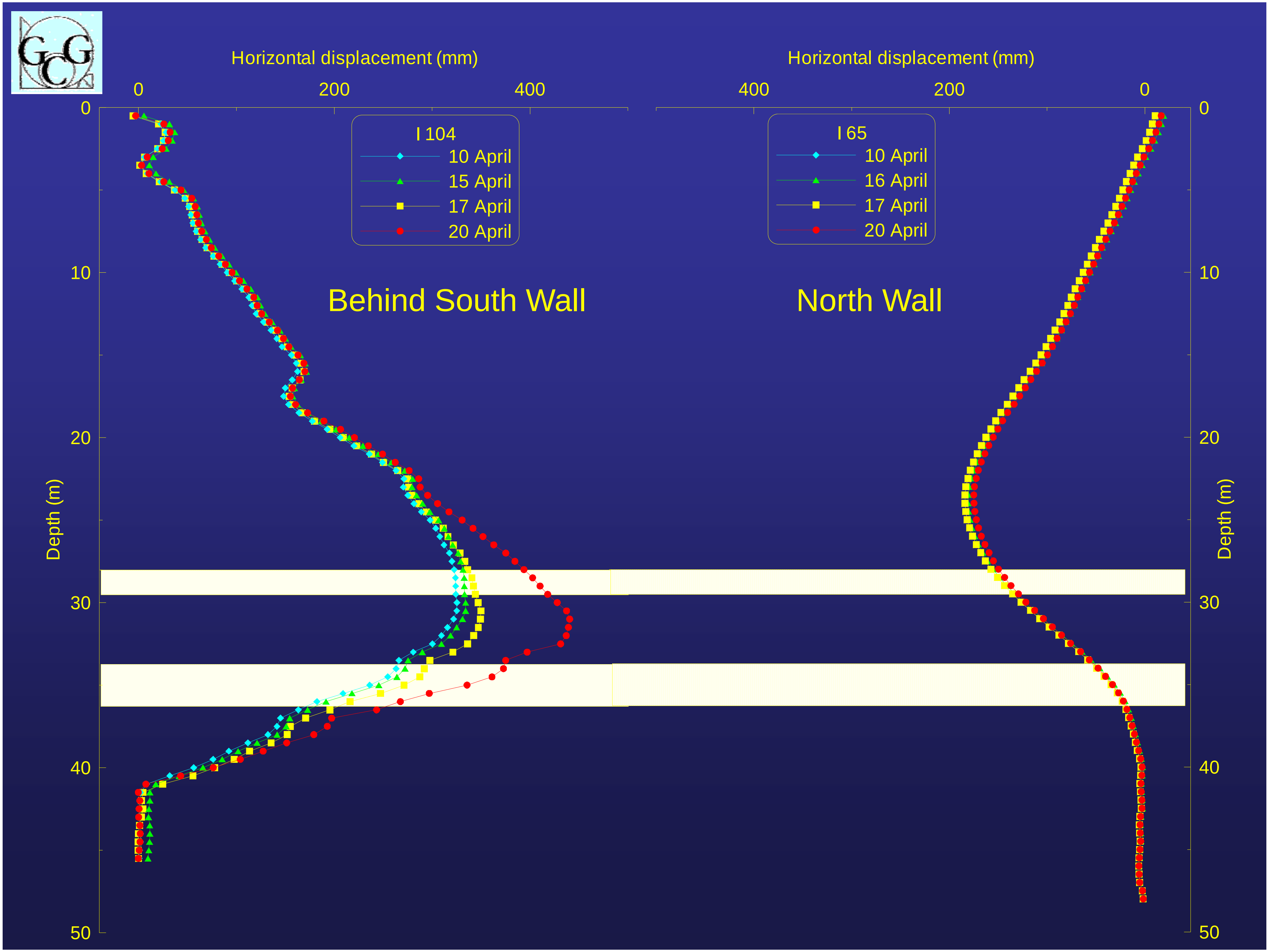
GCG
Horizontal displacement (mm)
0
200
400
I 104
10 April
Behind South Wall
Depth (m)
0
10
20
30
40
50
Horizontal displacement (mm)
400
200
0
I 65
10 April
North Wall
Depth (m)

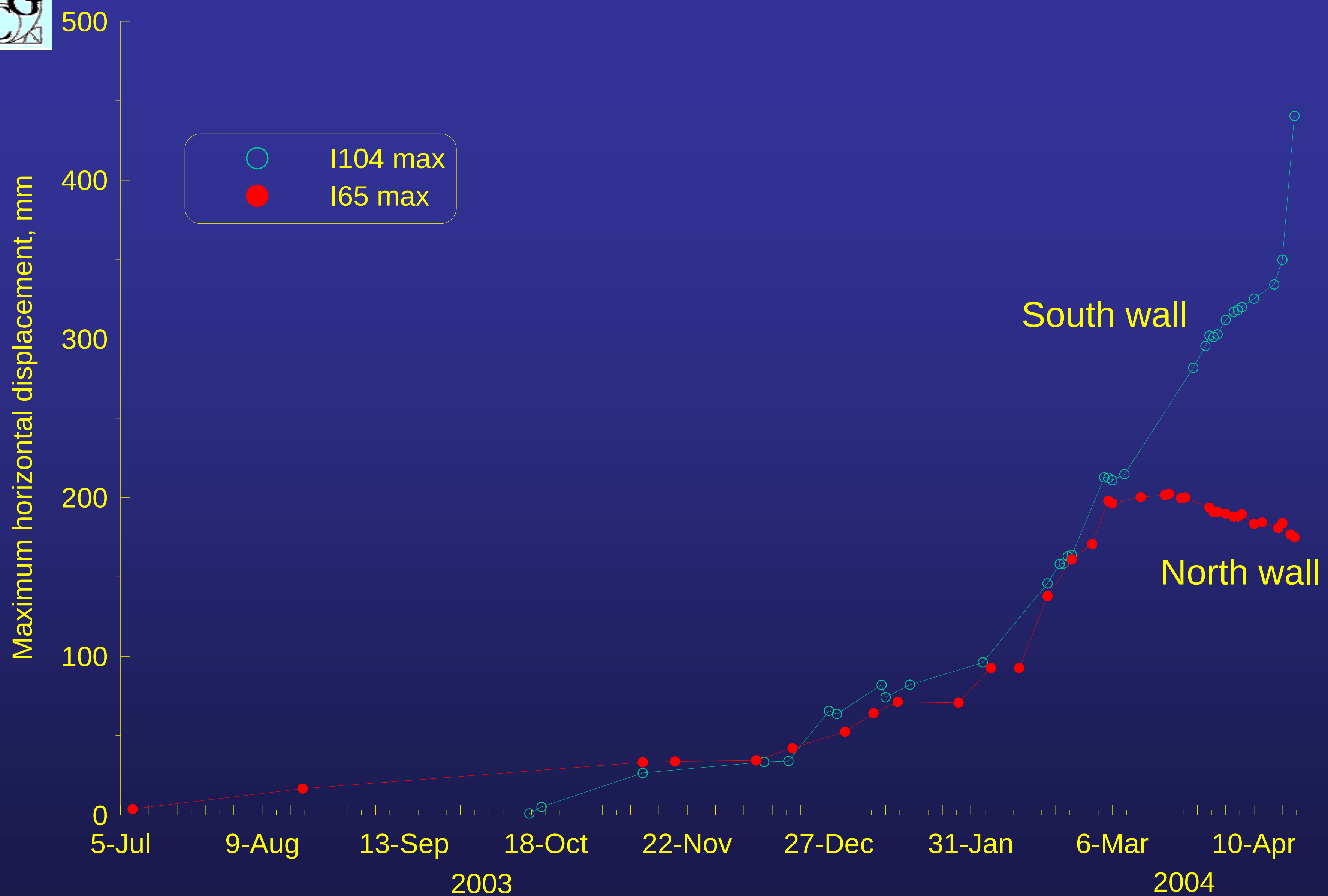


Horizontal displacement (mm)
0
200
400
Horizontal displacement (mm)
400
200
0
I 104
10 April
15 April
I 65
10 April
16 April
Behind South Wall
North Wall
Depth (m)
Depth (m)
0
10
20
30
40
50



Horizontal displacement (mm)
0
200
400
Horizontal displacement (mm)
400
200
0
Depth (m)
0
10
20
30
40
50
I 104
10 April
15 April
17 April
I 65
10 April
16 April
17 April
Behind South Wall
North Wall


GCG
Horizontal displacement (mm)
0
200
400
Horizontal displacement (mm)
400
200
0
0
10
20
30
40
50
Depth (m)
I 104
10 April
15 April
17 April
20 April
Behind South Wall
I 65
10 April
16 April
17 April
20 April
North Wall
Depth (m)



Comparison of inclinometer readings I65 and I104

- S338-9 stood for 8 days under load and was 20m from excavation front
- S335-9 stood for 2.5 days under load and was over 8m from excavation front
- Both S335-9S & S338-9N buckled within 10 minutes
- Load in S335-8 and S335-9 was almost constant between 18 April and initiation of collapse
- All C-channel connections failed downwards at both ends
- The south wall was pushing the north wall back

Key observations

The collapse


3.33pm


KOBELCO
3.33pm


3.34pm


3.34pm


3.34pm


3.41pm


3.46pm

Post-collapse investigations

Design errors

Errors

- Misinterpretation of BS5950 with regard to stiff bearing length
- Omission of splays

Effects

- Design capacity of strut-waler connection was 50% of required design capacity where splays were omitted

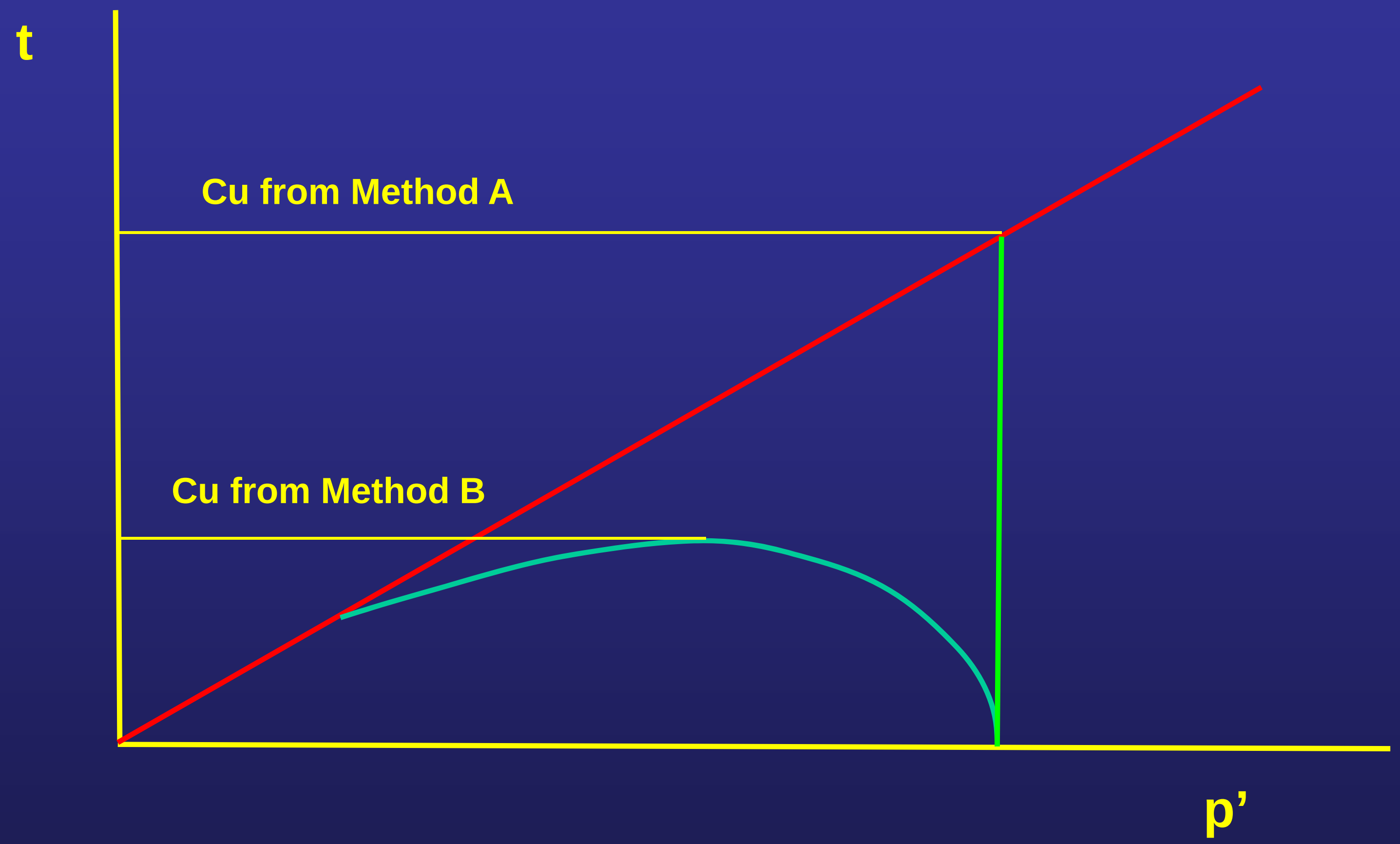
Errors in structural design of strut-waler connection

- Capacity based on BS5950:1990 = 2550 kN
- Average ultimate capacity based on physical load tests = 4100 kN
- Based on mill tests, 95% of connections had capacity of 3800 kN- 4400kN
- Predicted 9th level strut load in 2D analyses which ignored bored piles was close to ultimate capacity therefore collapse was considered by the COI to be inevitable

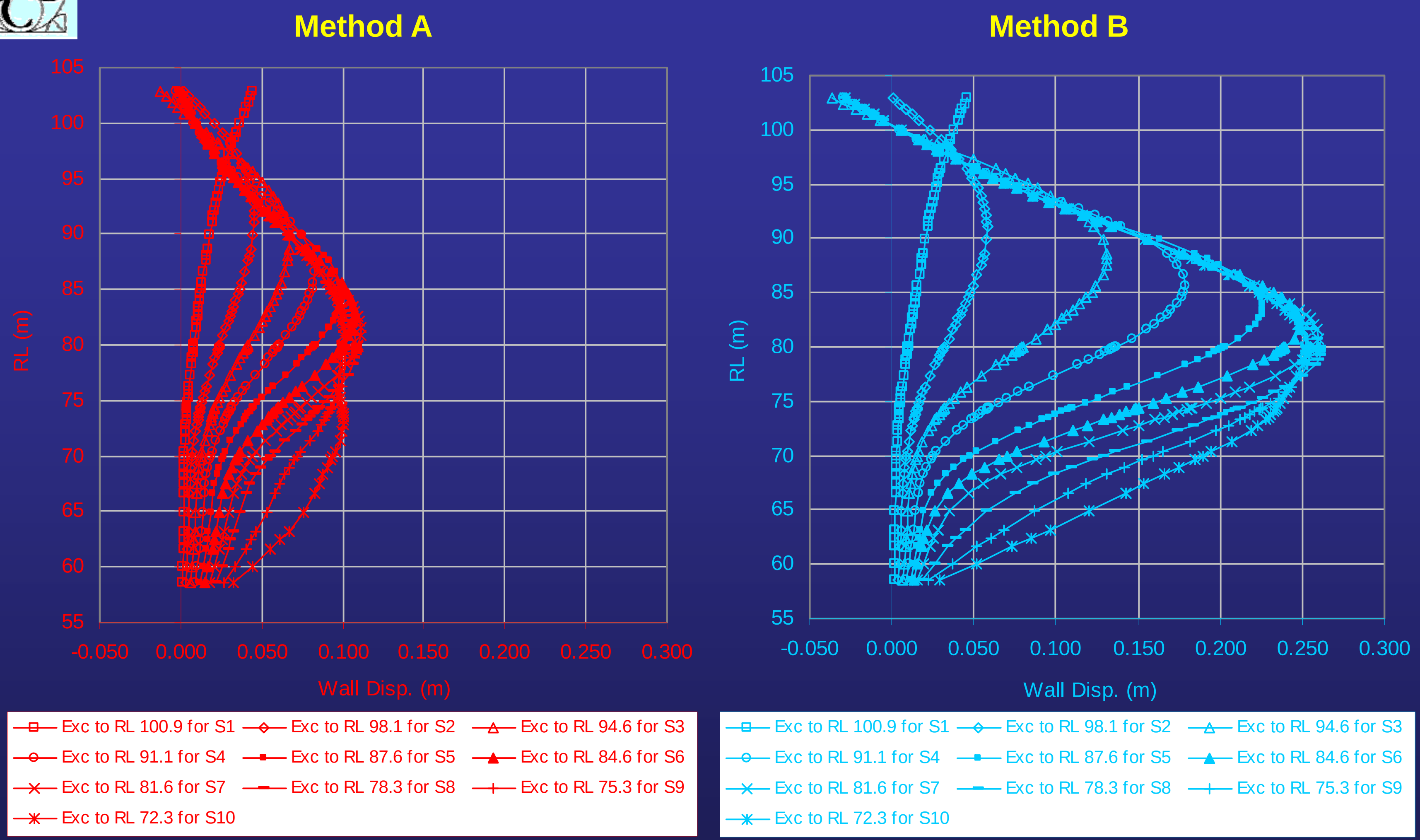
Inevitability of collapse

- Method A and Method B refer to two alternative ways of modelling undrained soil behaviour in Plaxis (Pickles, 2002)
- Method A is an effective stress analysis of an undrained problem
- Assumes isotropic elastic behaviour and a Mohr-Coulomb failure criterion
- As a result mean effective stress p’ is constant until yield
- Method A was being applied to marine clays which were of low over-consolidation or even under-consolidated because of recent reclamation
- Method B is a total stress analysis

Methods A and B

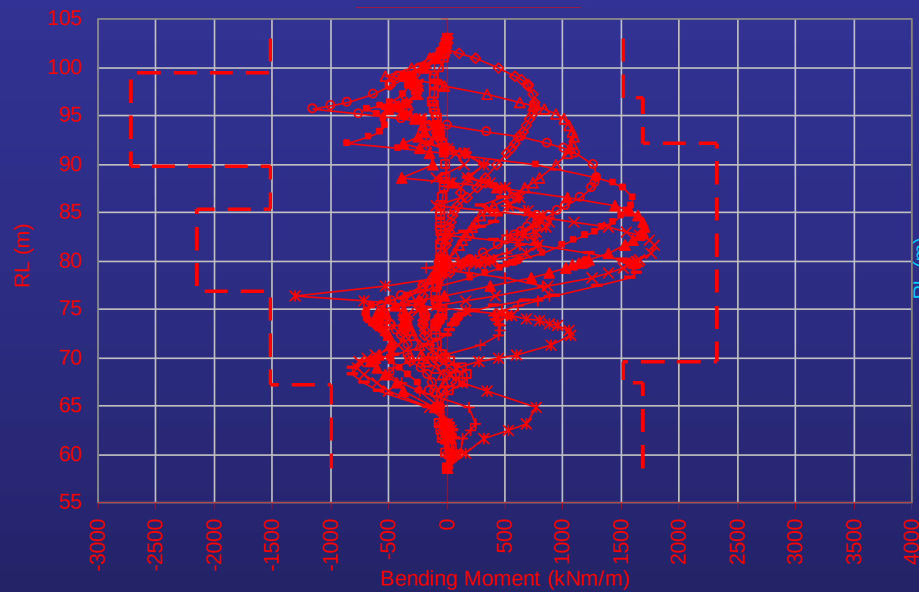


The shortcomings of Method A

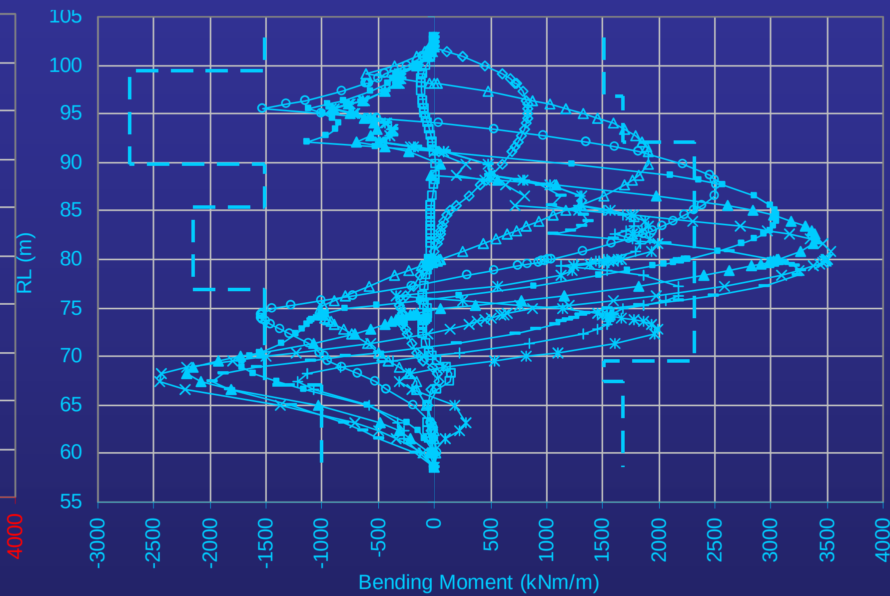


M3 - South Wall Displacement
Method A versus Method B

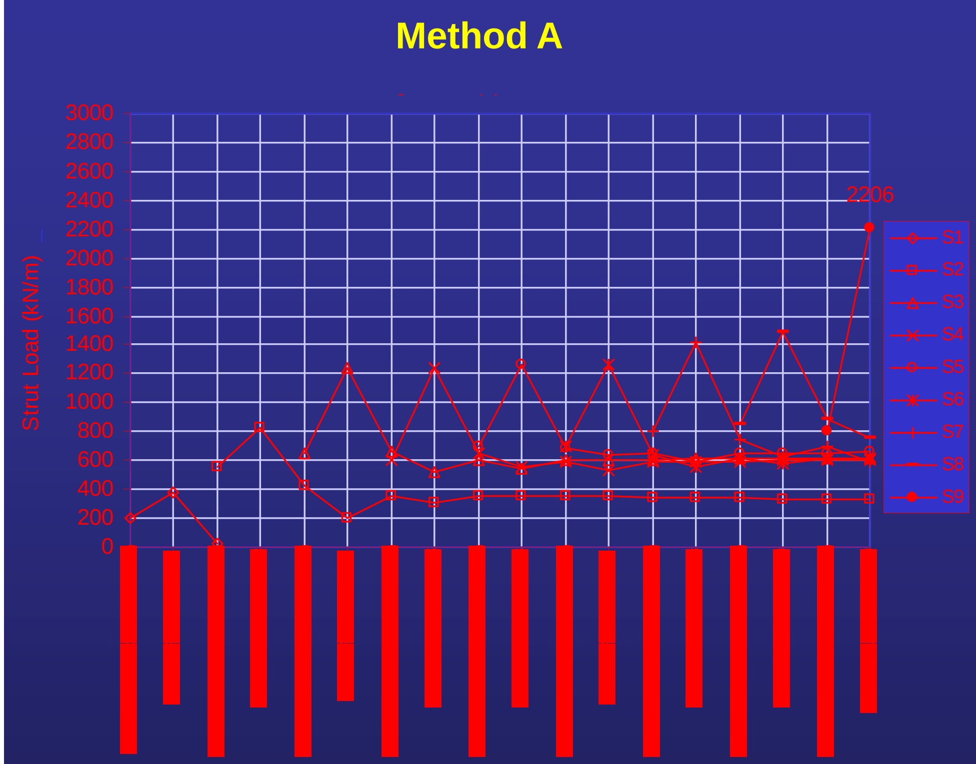
Method A

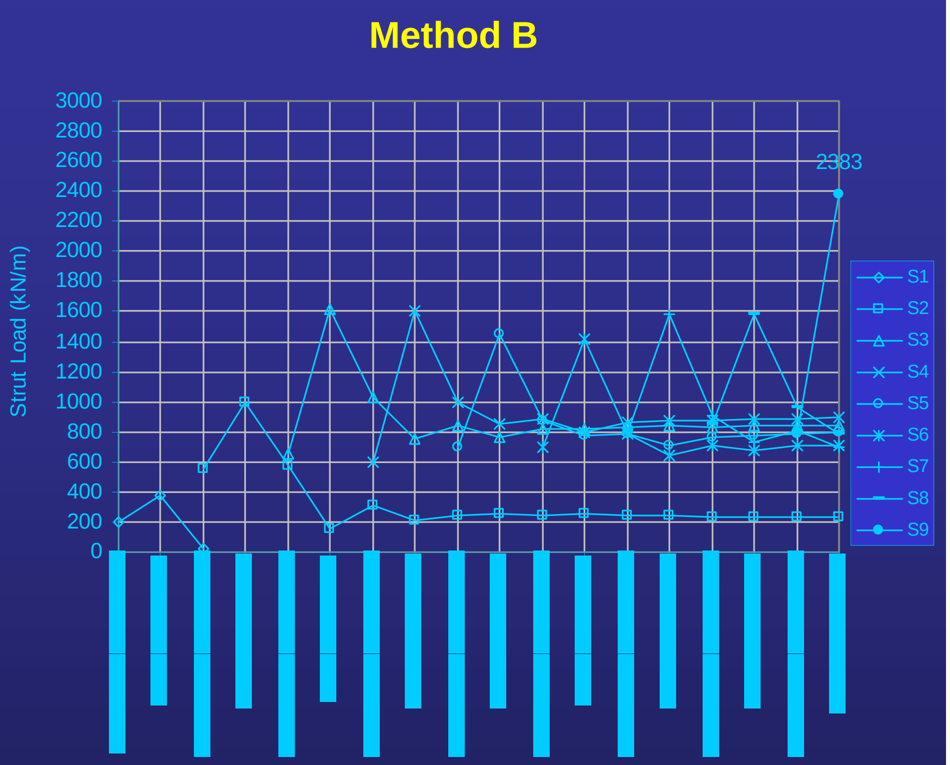


Method B



M3 - South Wall bending moments
Method A versus Method B





M3 – strut forces
Method A versus Method B

- Method A over-estimates the undrained shear strength of normally and lightly overconsolidated clays

- Its use led to a 50% under-estimate of wall displacements and of bending moments and an under-estimate of the 9th level strut force of 10%

- The larger than predicted displacements mobilised the capacity of the JGP layers at an earlier stage than predicted

Method A

- Structural design errors
- Removal of splays at some strut locations
- Introduction of C-channel waler connection detail
- Use of Method A in soil-structure interaction analysis
- Collapse was an inevitable consequence of the design errors which led to the applied loads on the struts increasing with time and equalling the capacity of the strut-waler connection

COI view on the principal causes of the collapse

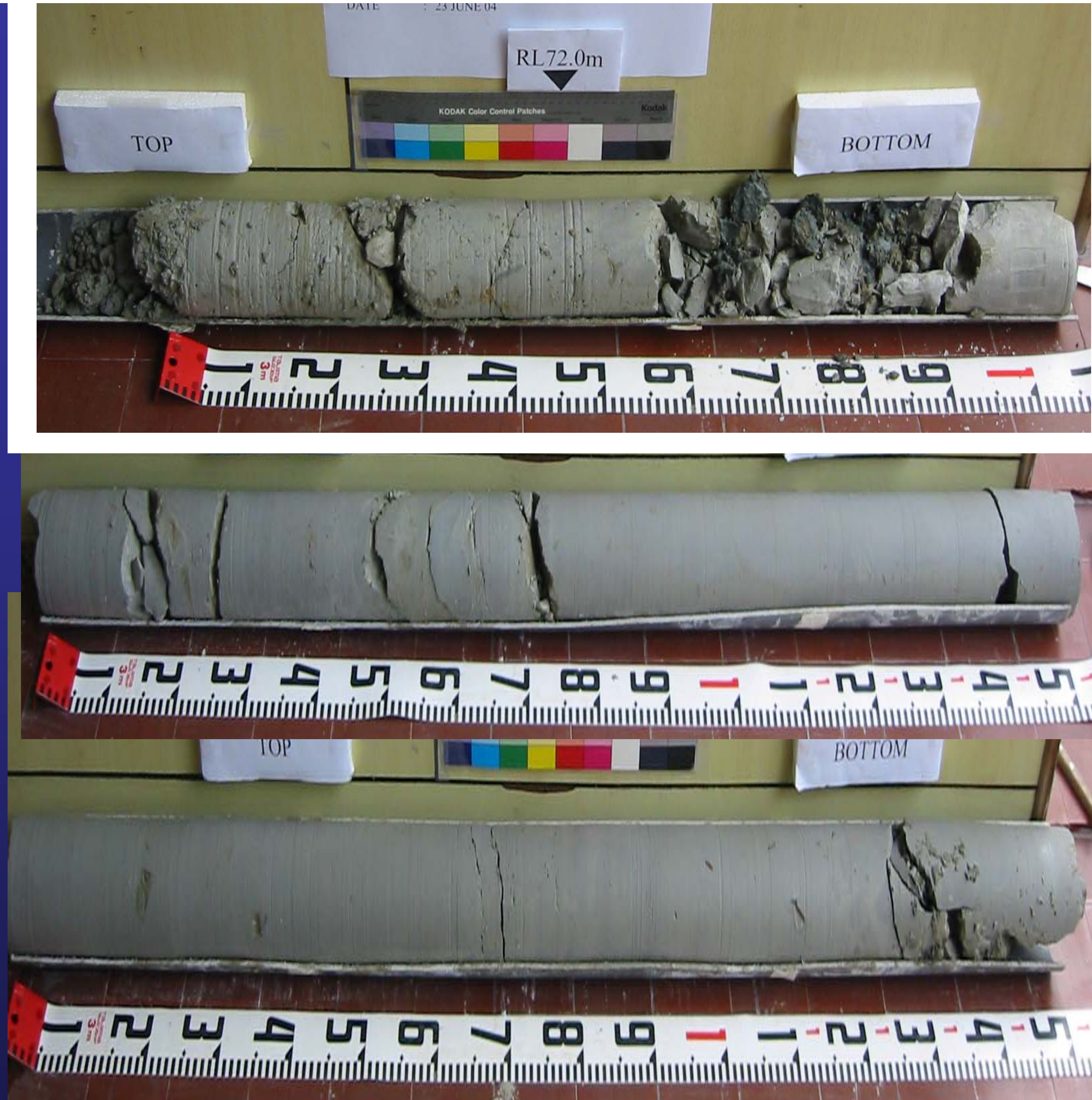
- Structural design errors
- Removal of splays at some strut locations
- Introduction of C-channel waler connection detail
- Use of Method A in soil-structure interaction analysis
- Collapse was an inevitable consequence of the design errors which led to the applied loads on the struts increasing with time and equalling the capacity of the strut-waler connection

COI view on the principal causes of the collapse

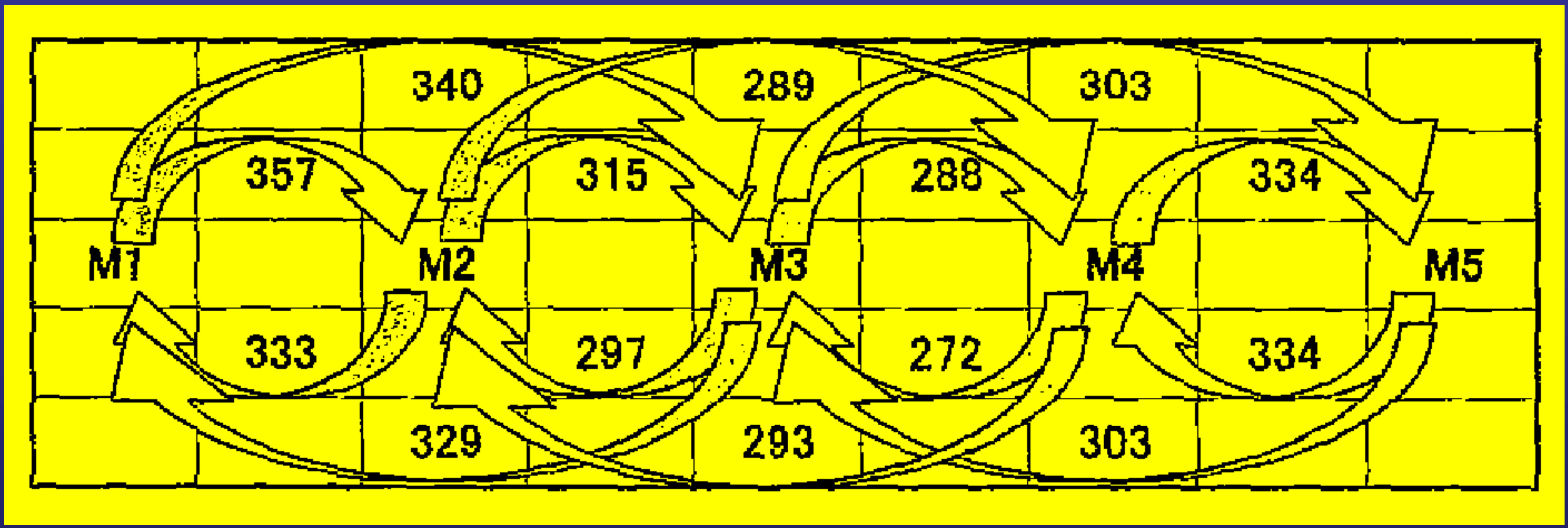
Post-collapse investigations

Jet grout

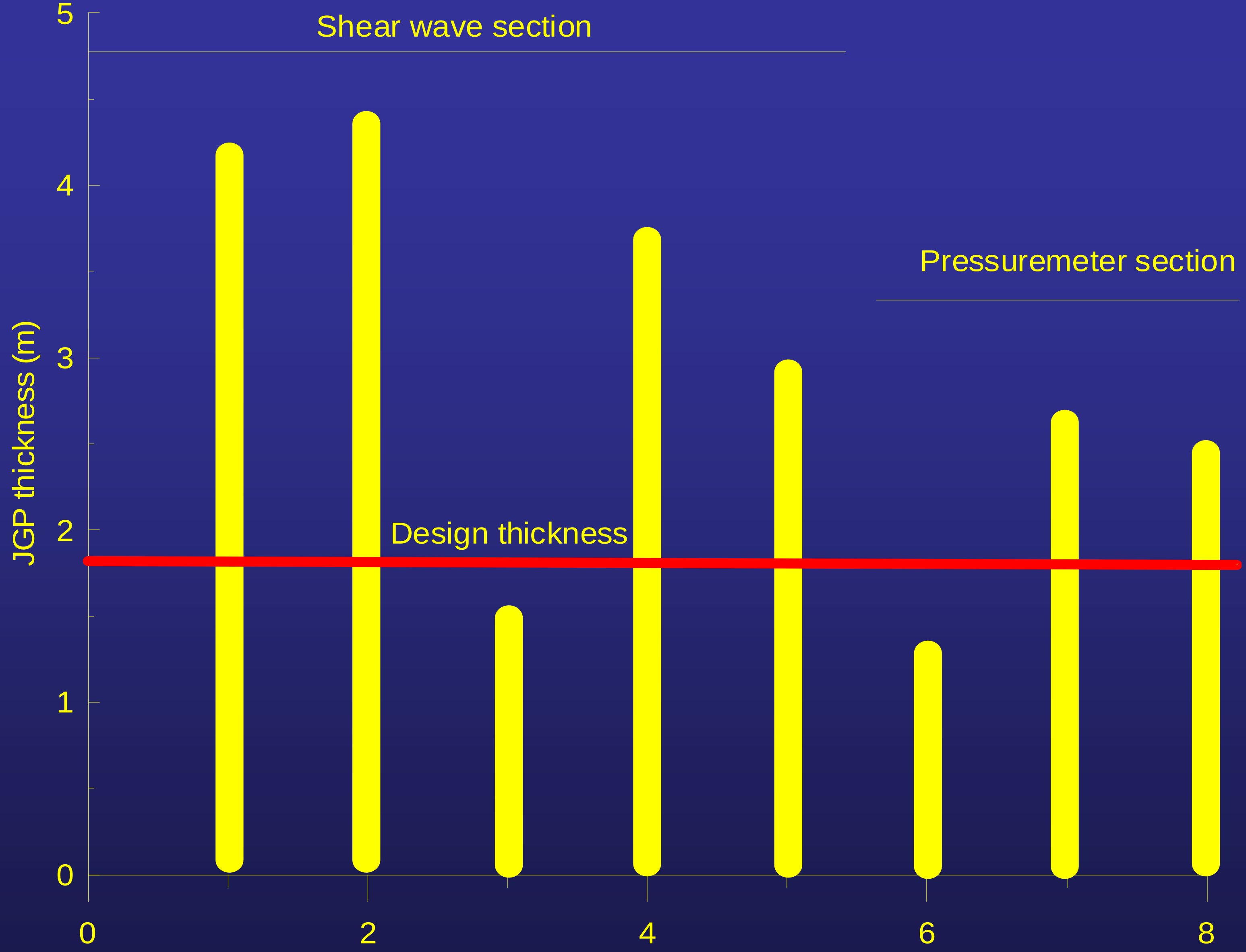

Excavation of sacrificial JGP in Type H



JGP quality in 100mm cores from borehole M1 in Type K



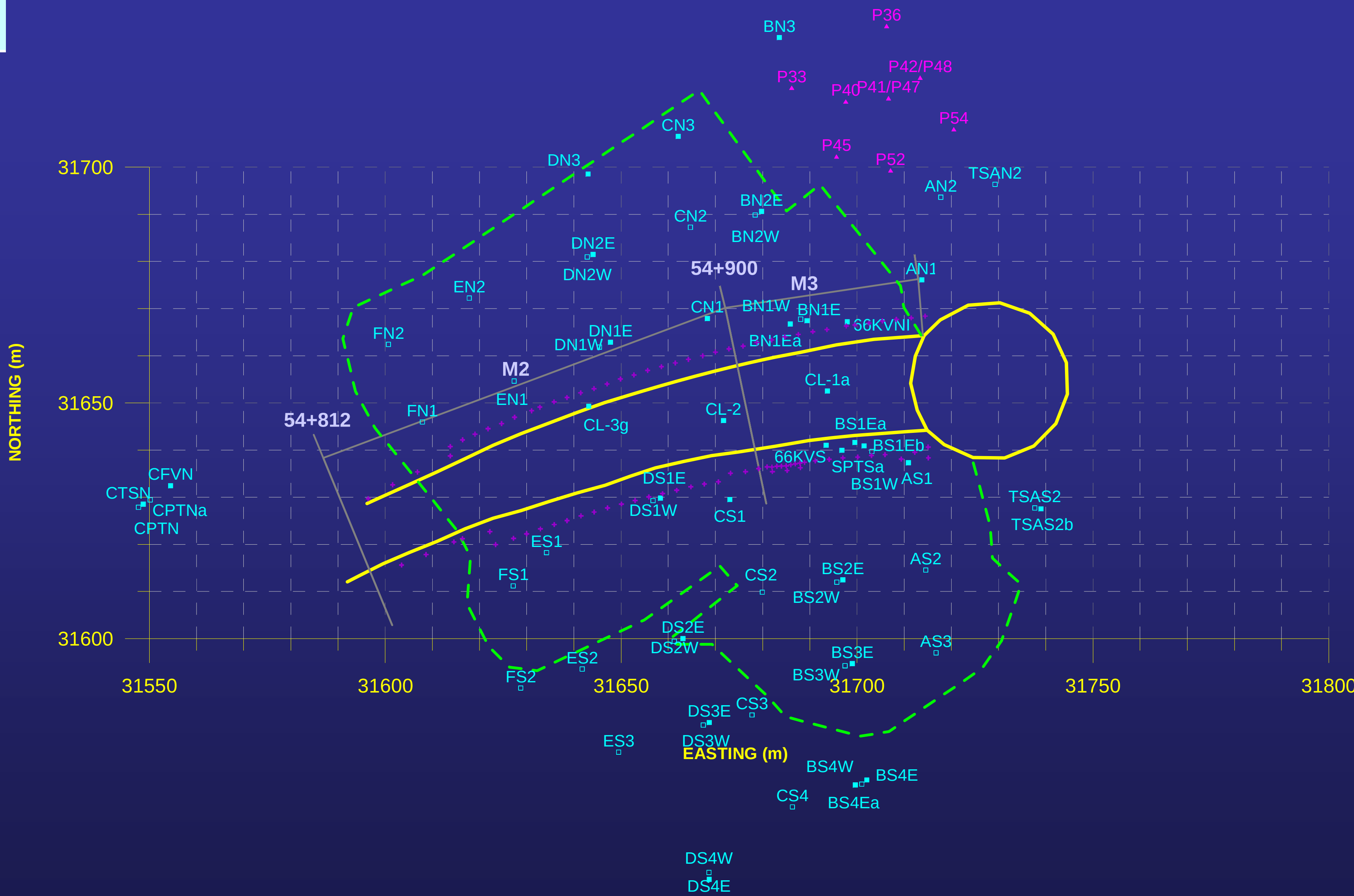
Shear wave velocity measurements in JGP at Type K



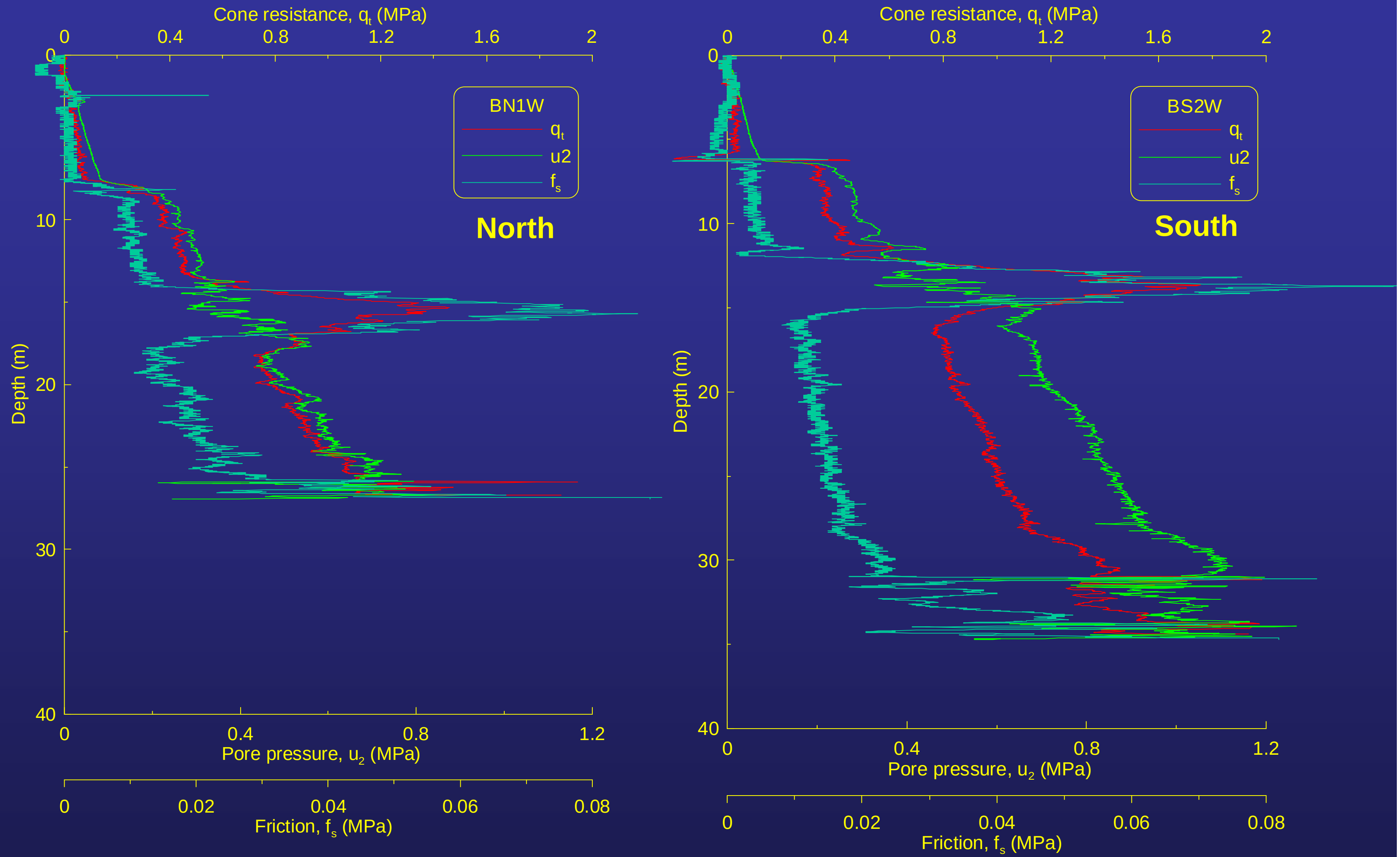
Thicknesses of JGP in Type K

Post-collapse investigations

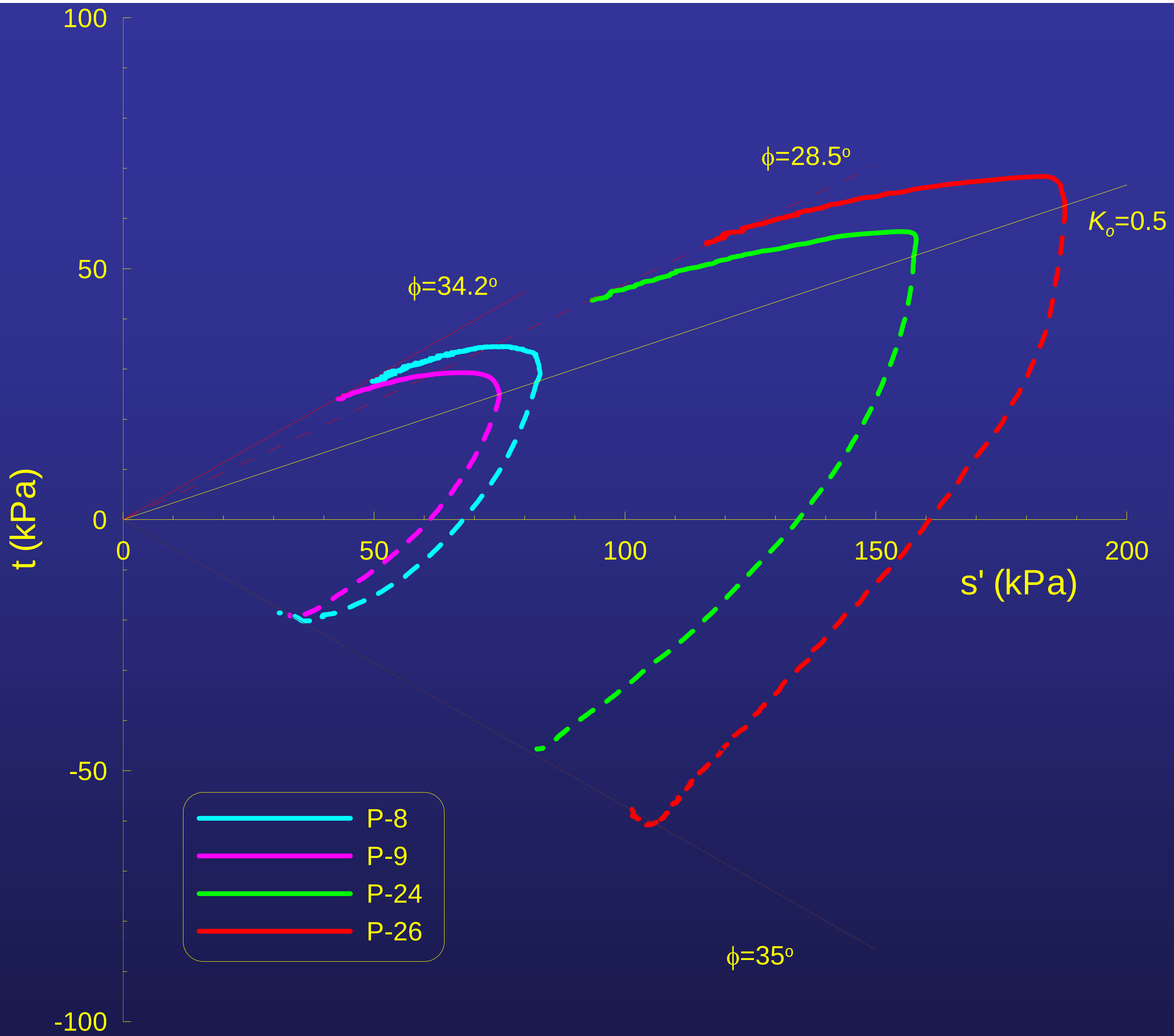
Ground conditions and soil properties



Post-collapse ground investigation

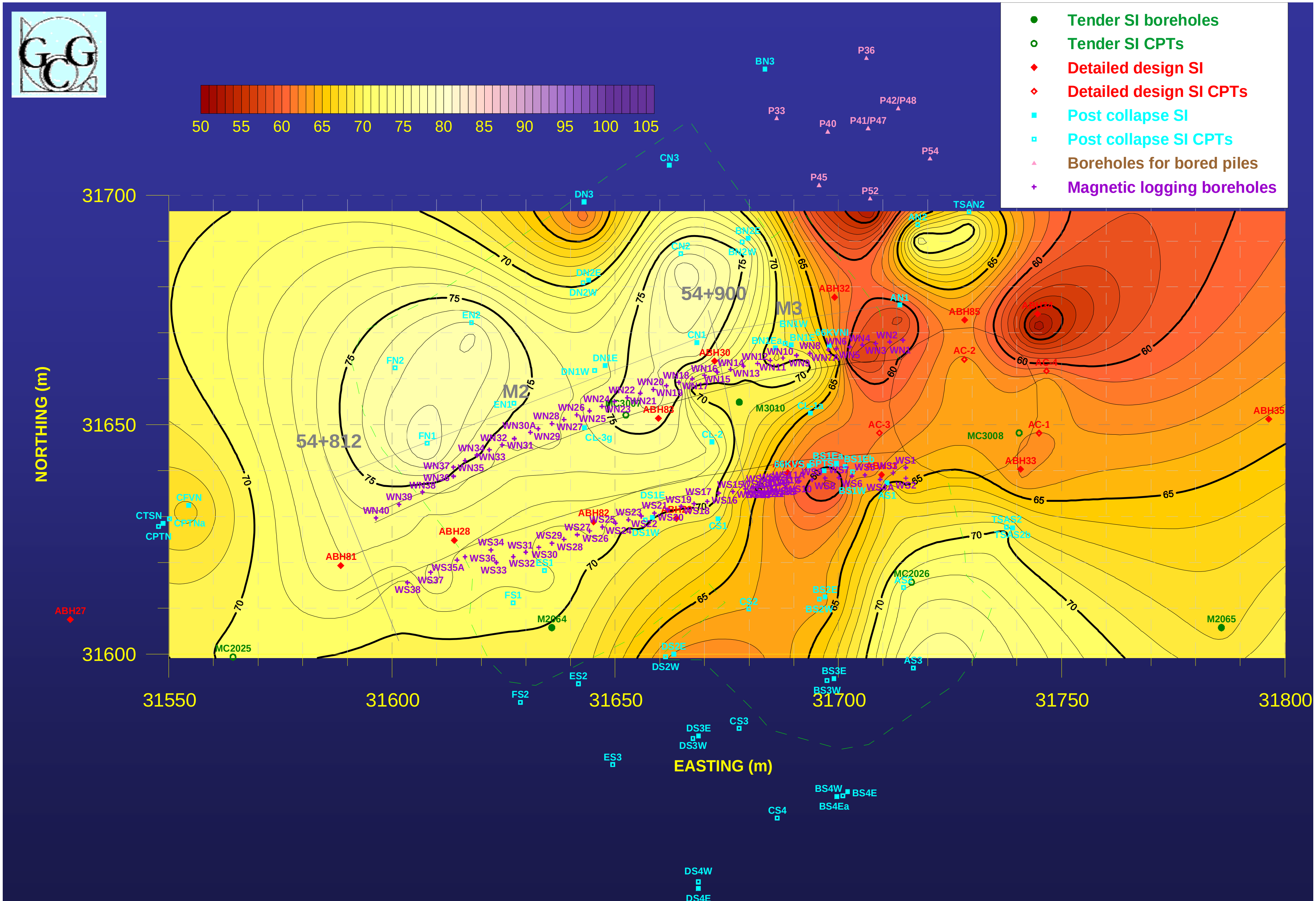


CPT profiles north and south of collapse area

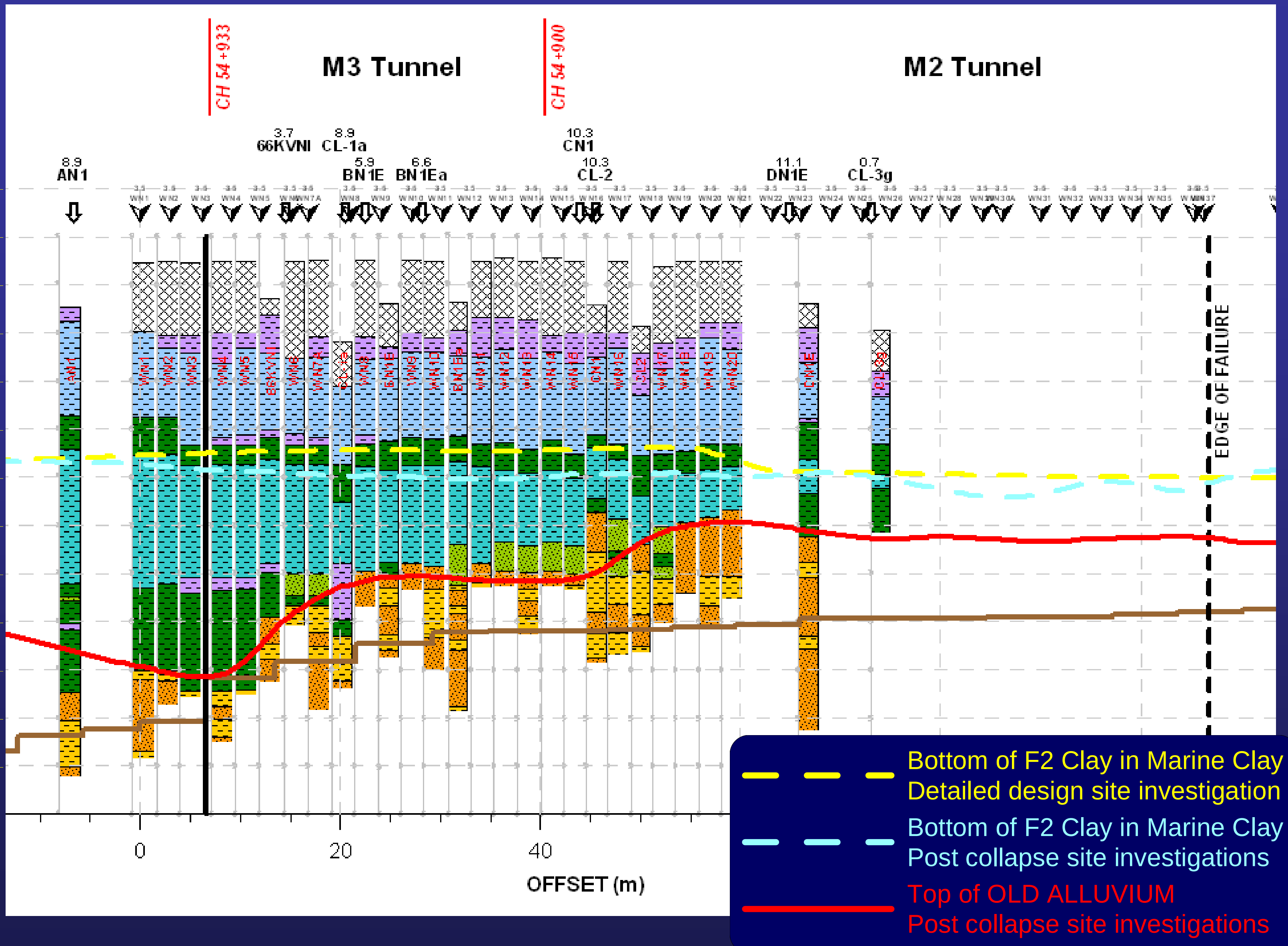


Kisojiban's CAU tests on

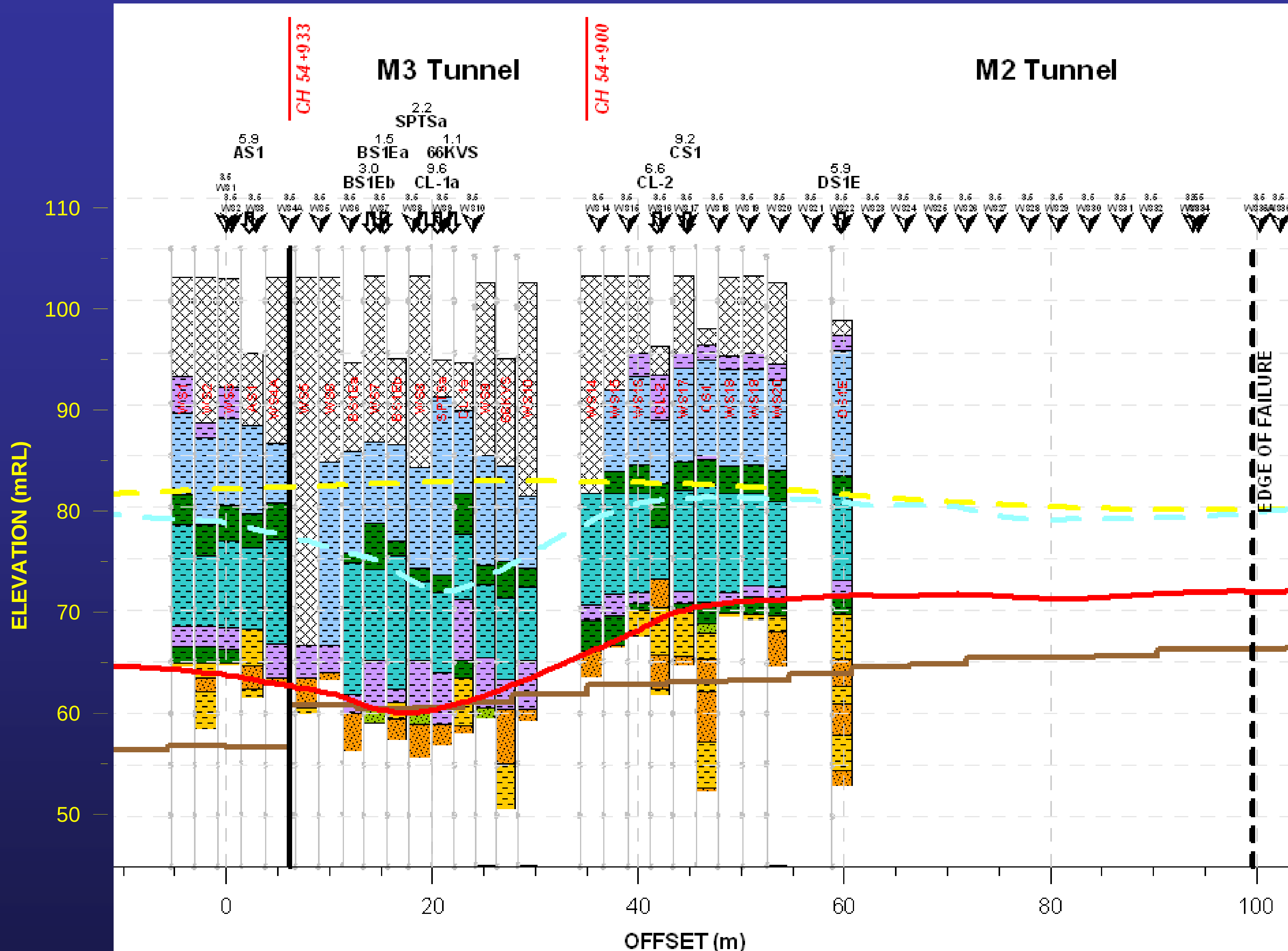
Upper and Lower Marine Clay



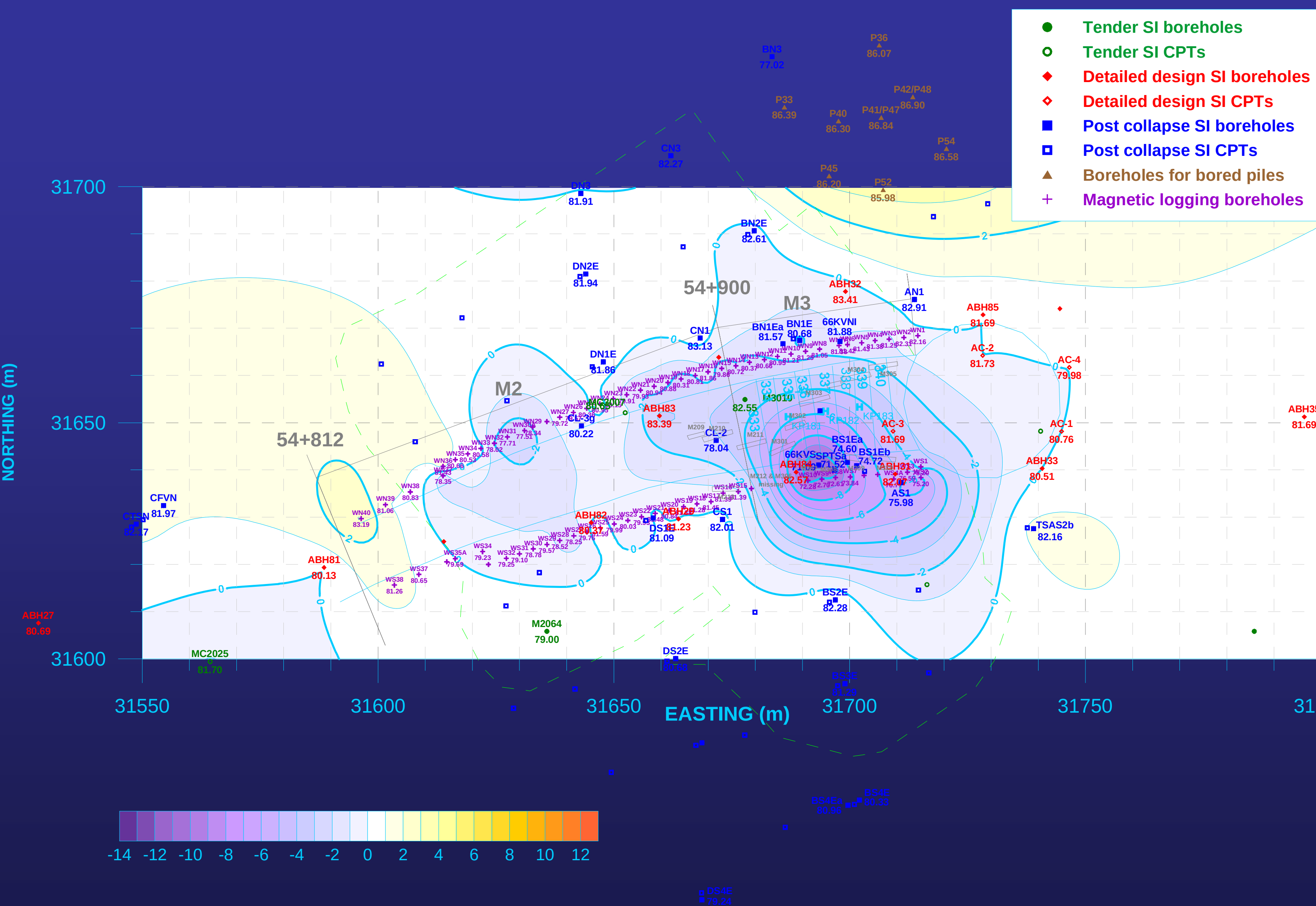
Evidence for a buried valley in the Old Alluvium



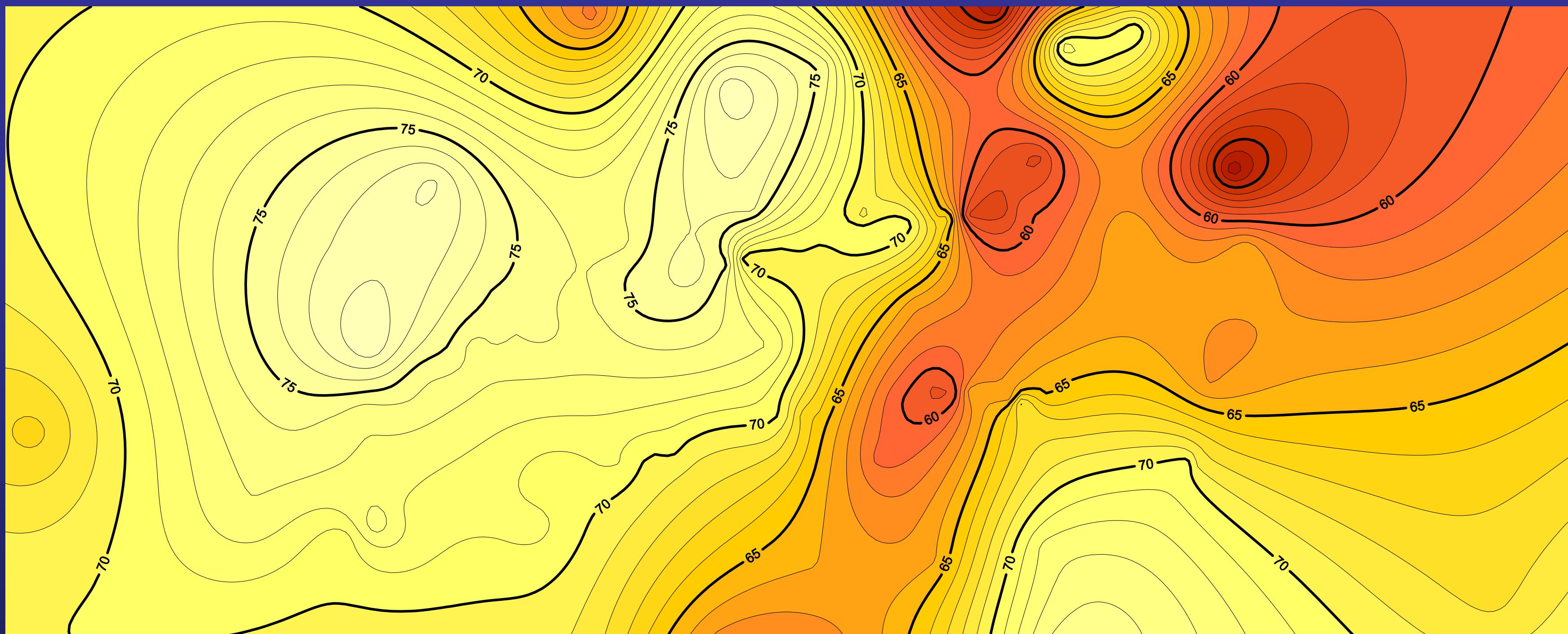
Section along north wall



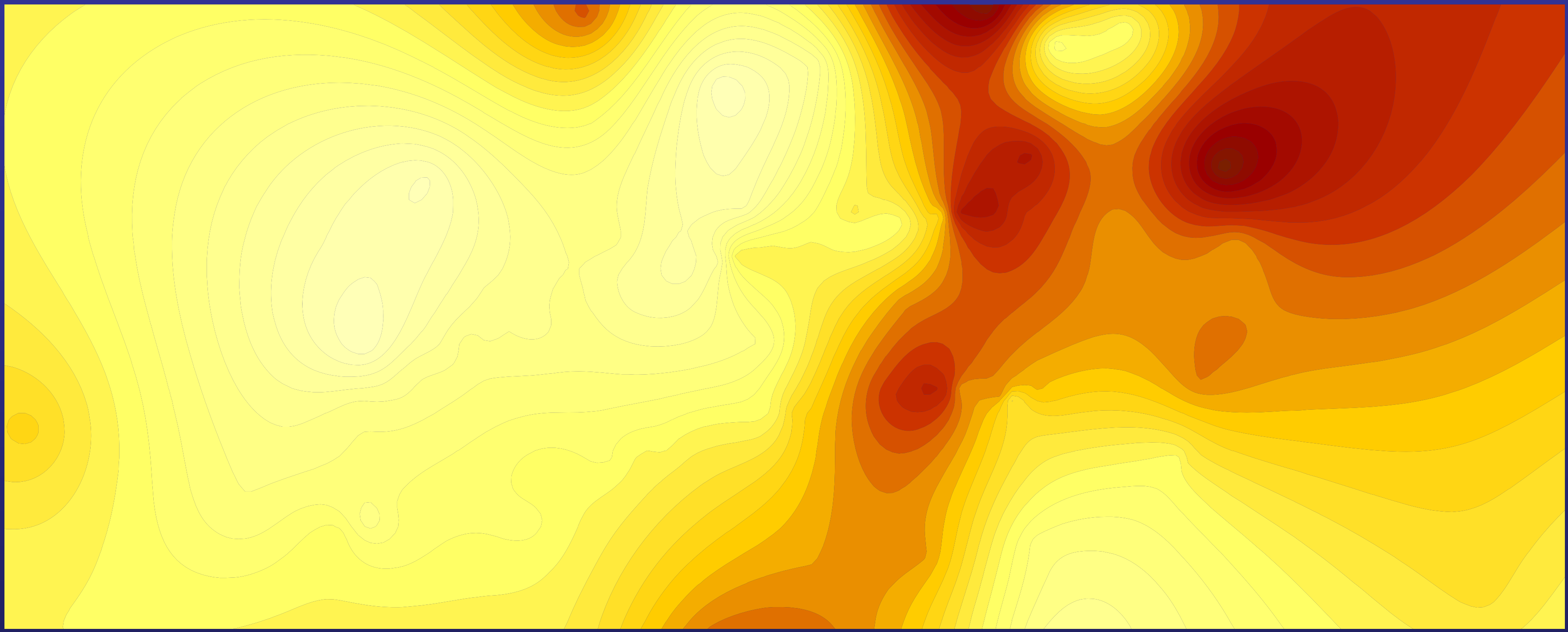
Section along south wall



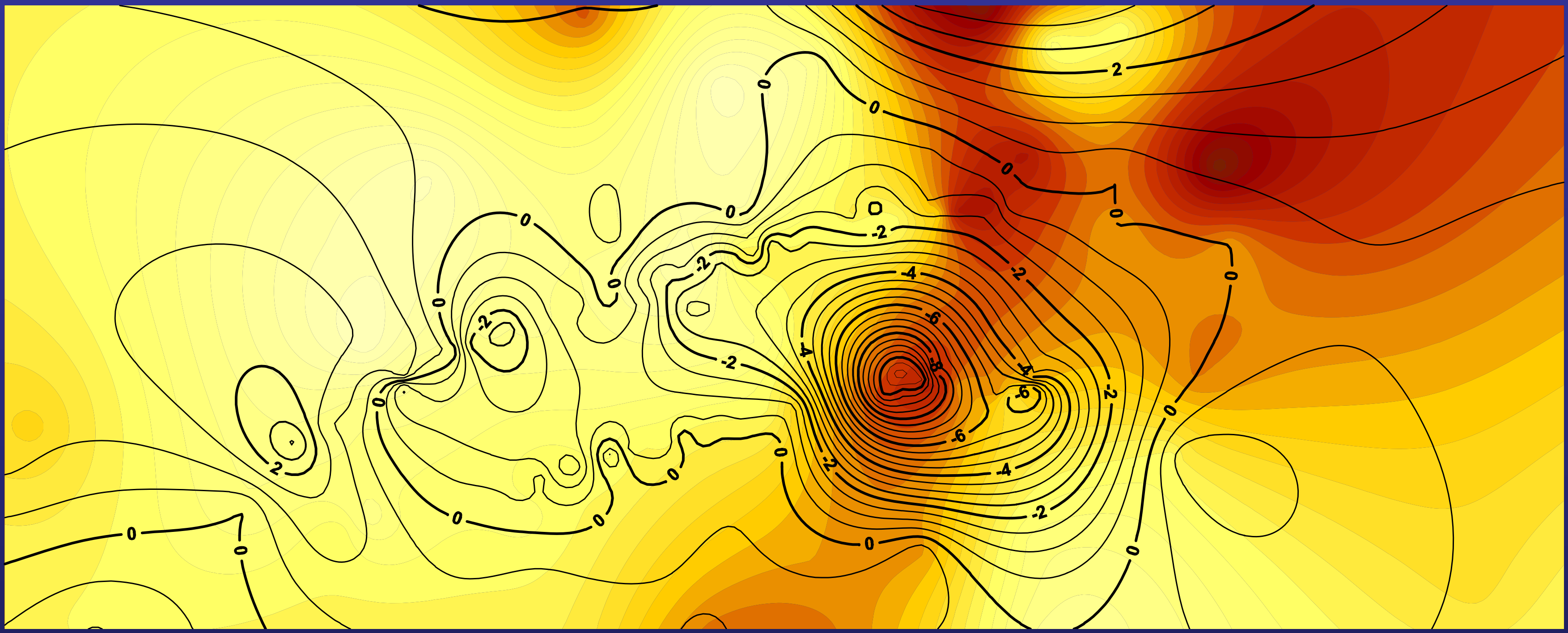
Distortion to upper F2 layer caused by the collapse



Buried valley in the Old Alluvium



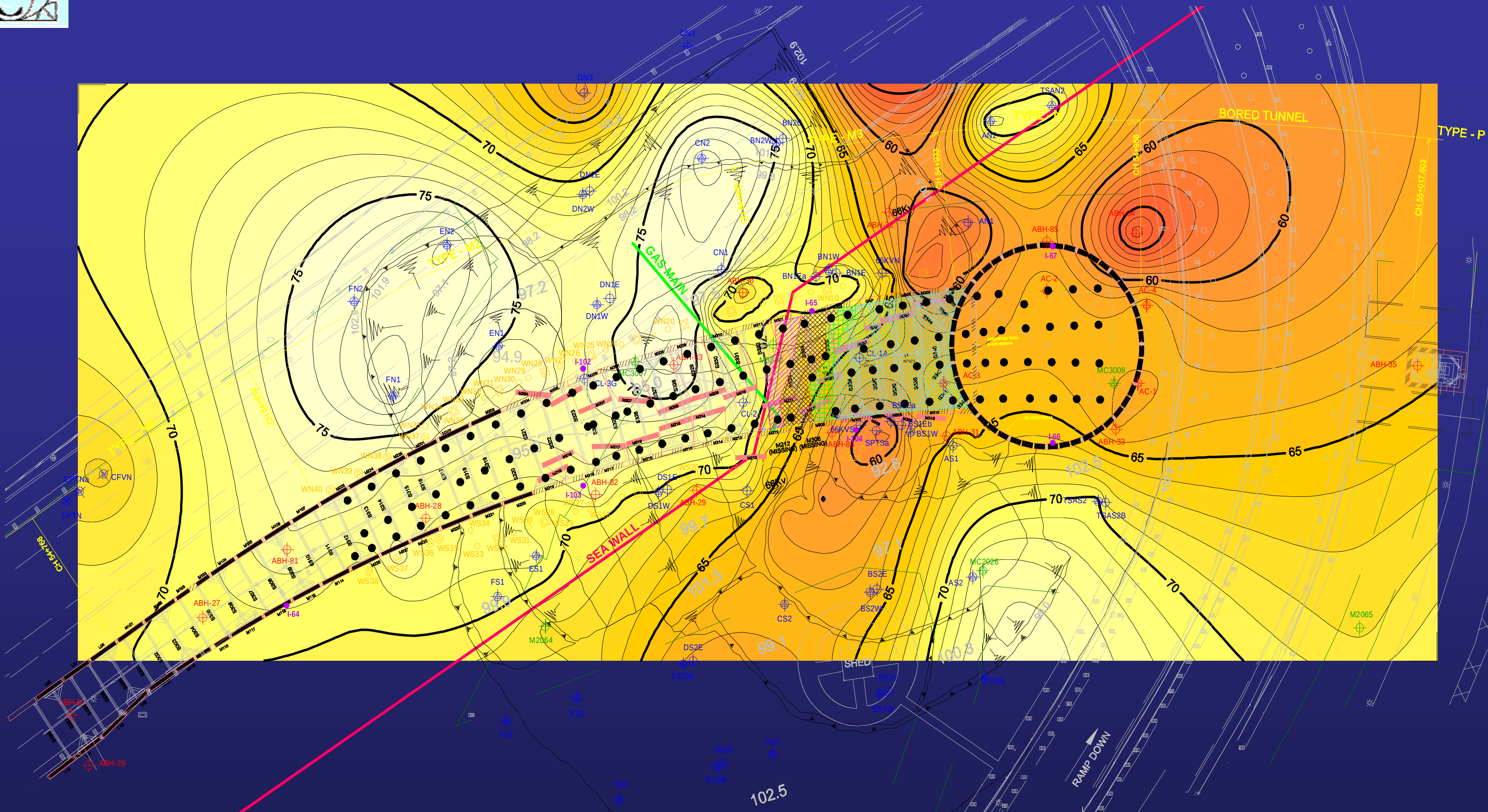
Buried valley in the Old Alluvium



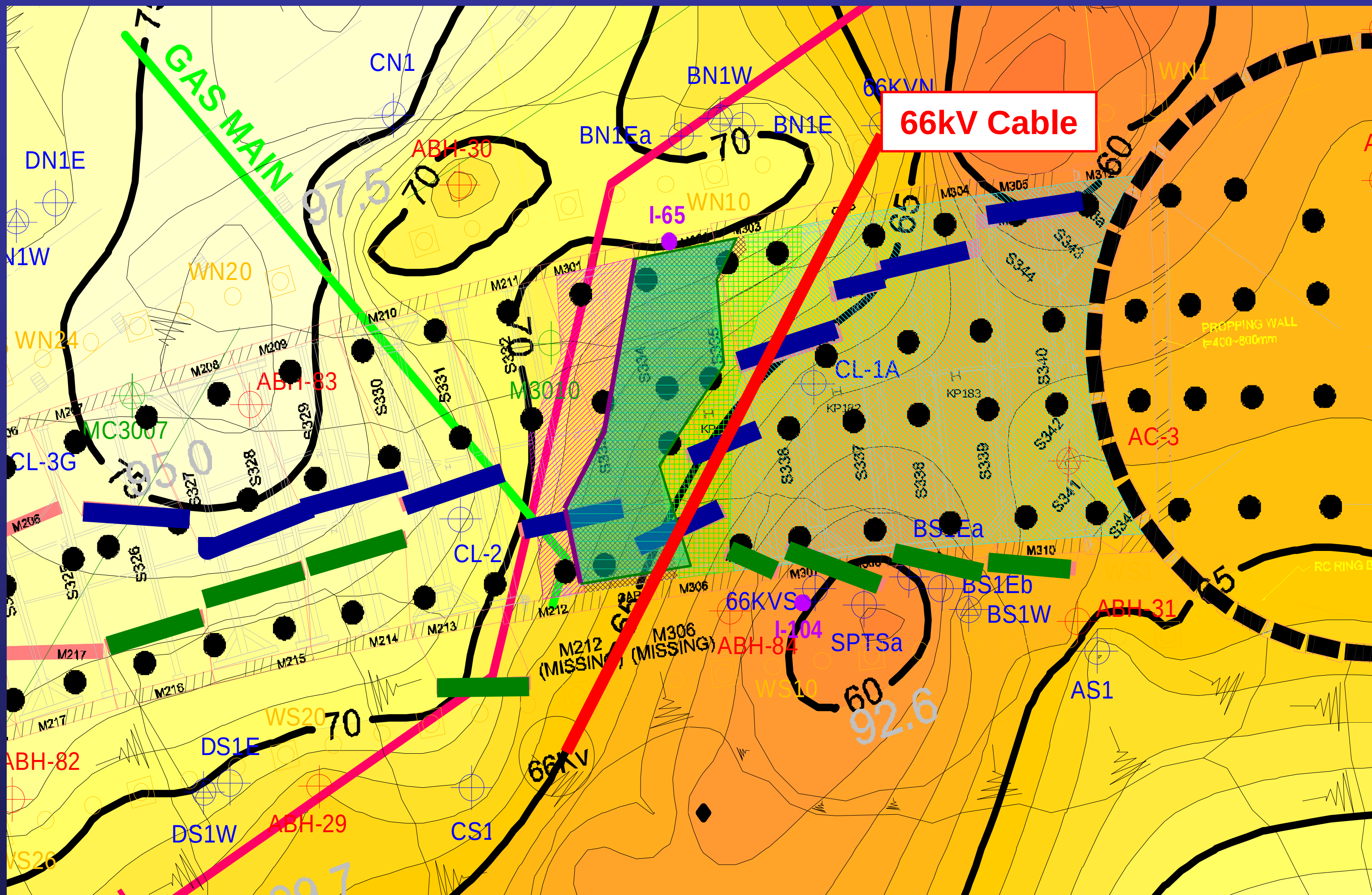
Coincidence between buried valley and distortion to upper F2 layer

- There was a buried valley crossing the site of the collapse diagonally from south-west to north-east
- The presence and setting of the buried valley explain the asymmetric conditions and the different collapse on the north and south sides
- The buried valley coincides with the major ground distortion on the south side and was clearly influential in the collapse
- Below the Lower Marine Clay the buried valley was infilled with estuarine organic clays on the south side and fluvial clays on the north side
- Gas exsolution almost certainly occurred in the deep organic clays as a result of stress relief, reducing their strength further

The buried valley



Post-collapse geometry and sequence of excavation superimposed on buried valley



Sequence of excavation in relation to buried valley

- **The buried valley was crossed without collapse developing**
- Strut forces would have been a maximum in the buried valley and would have varied across the valley
- The collapse was not, therefore, inevitable
- An external influence (trigger) is required to explain the timing of the collapse and why it occurred after crossing the buried valley

Significance of crossing the buried valley without collapse

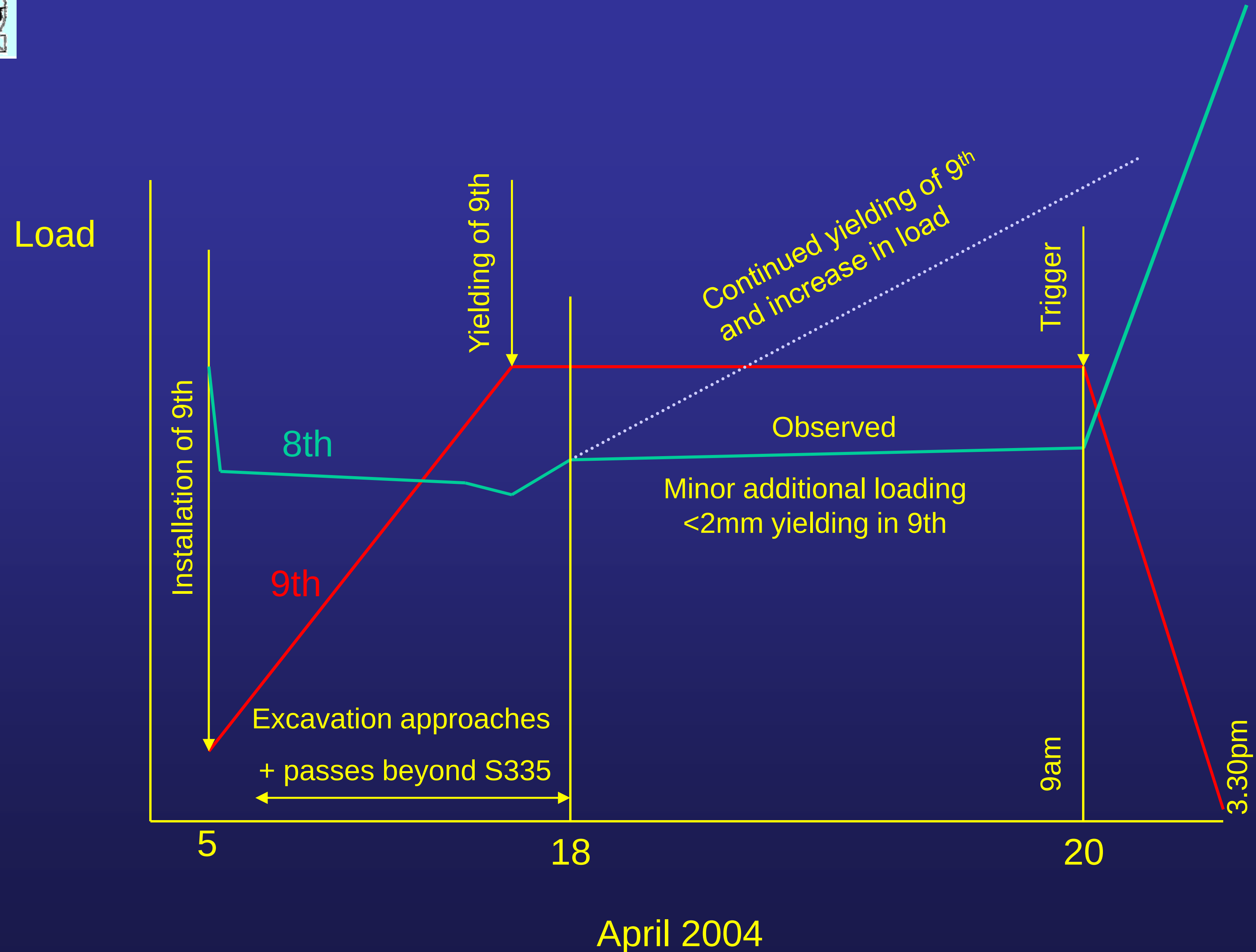
- S338-9 stood for 8 days under load and was 20m from excavation front
- S335-9 stood for 2.5 days under load and was over 8m from excavation front
- Both S335-9S & S338-9N buckled within 10 minutes
- Load in S335-8 and S335-9 was almost constant between 18 April and initiation of collapse
- All C-channel connections failed downwards at both ends
- The south wall was pushing the north wall back

Key observations

- 3D effects cannot explain why the collapse was initiated at 9am – S338 was 20-24m from the excavation face and had stood for 8 days without distress, S335 was 8-12m from the excavation face

- Time effects cannot explain why the collapse was initiated at 9am – there was no evidence of load increases in the monitoring

Potential triggers


Load
Installation of 9th
Yielding of 9th
Continued yielding of 9th
and increase in load
Trigger
8th
9th
Observed
Minor additional loading
<2mm yielding in 9th
Excavation approaches
+ passes beyond S335
9am
3.30pm
5
18
20
April 2004

Post-collapse investigations

Back analyses of the collapse

- Analyses by Dr Felix Schroeder and Dr Zeljko Cabarkapa using Imperial College Finite Element Program (ICFEP)
- 2D section through M307 (I104) and M302 (I65)
- Bored piles are not modelled (enhanced JGP)
- Upper and Lower Marine Clays, F2 and lower Estuarine Clay modelled using Modified Cam Clay
- Coupled consolidation
- Fill and OA sand modelled using Mohr Coulomb. OA-CZ clay/silt modelled as Tresca

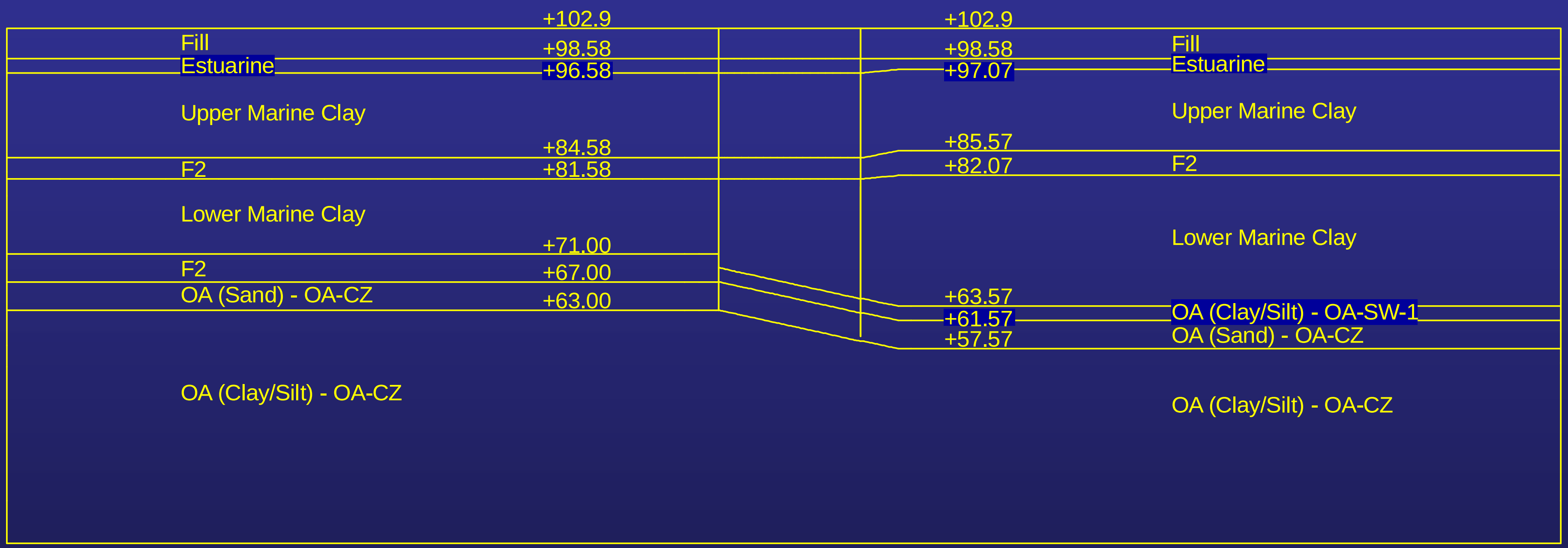
Geotechnical analyses

- Wall EI reduced to allow for cracking, based on reinforcement layout
- Bending moment capacities set according to reinforcement layout and ultimate strengths of steel and concrete as supplied
- JGP treated as brittle material
- 9th level strut capacity set and strut allowed to strain soften 72 hours after excavation to 10th level.

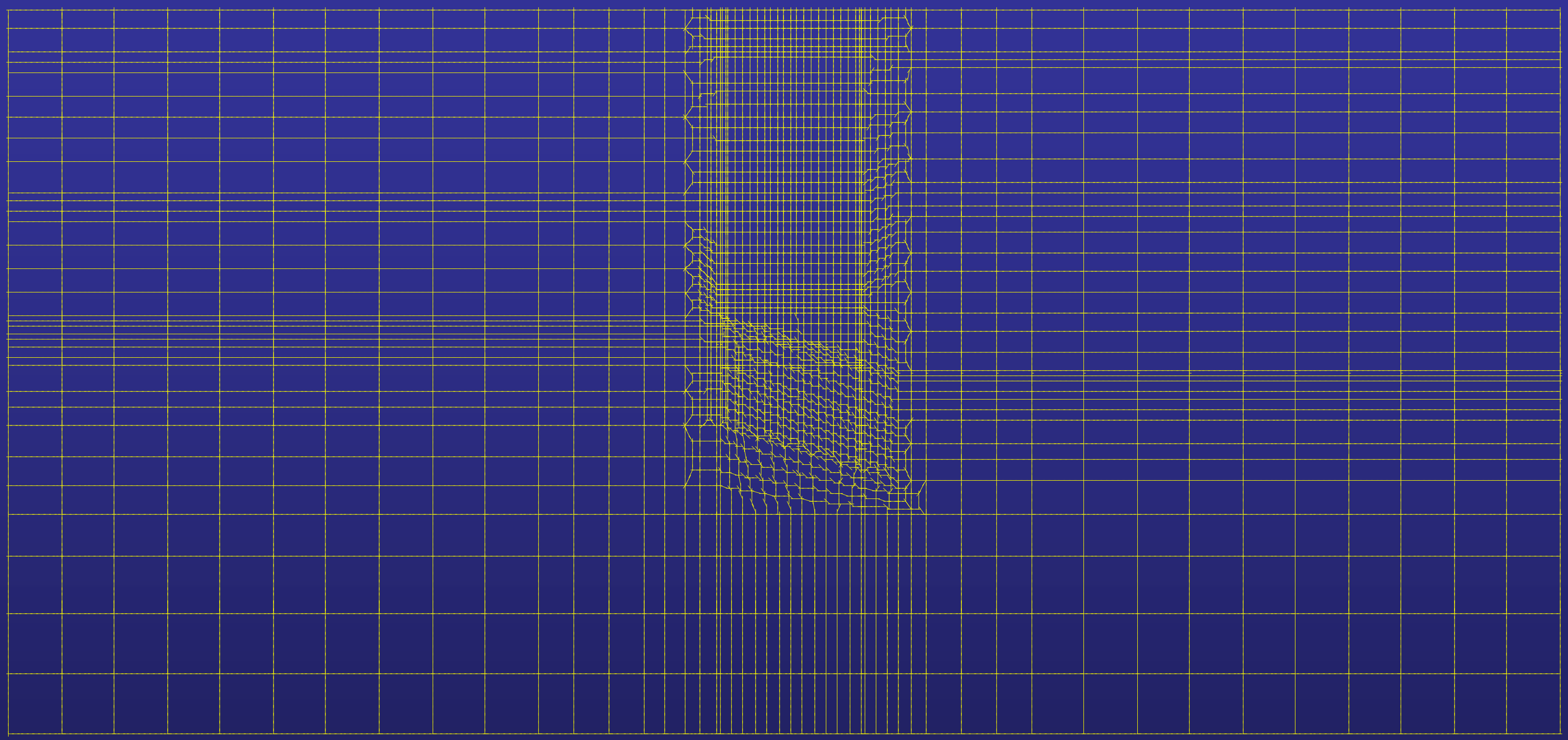
Geotechnical analyses

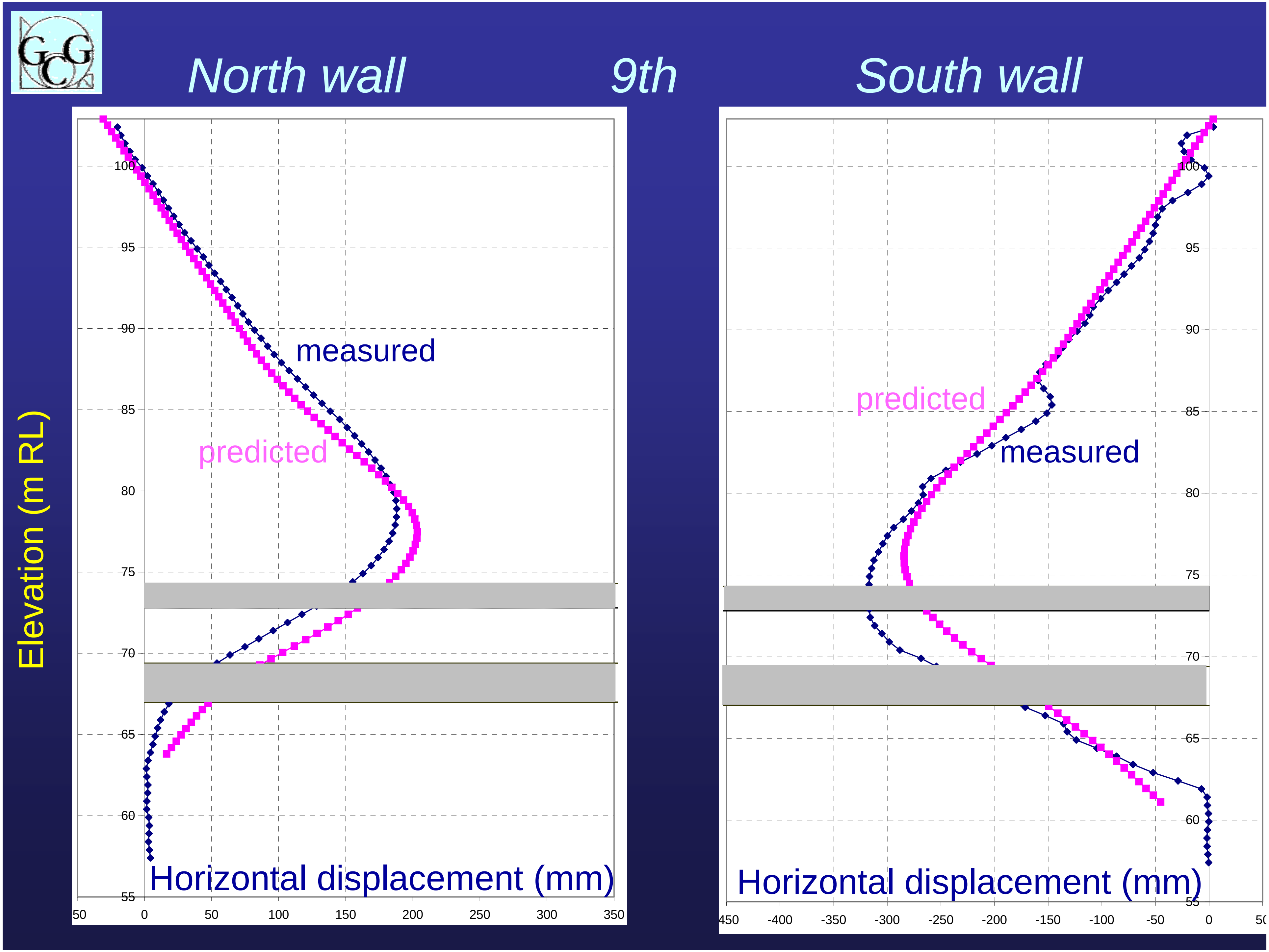
North

South



Stratigraphy assumed for ICFEP analyses

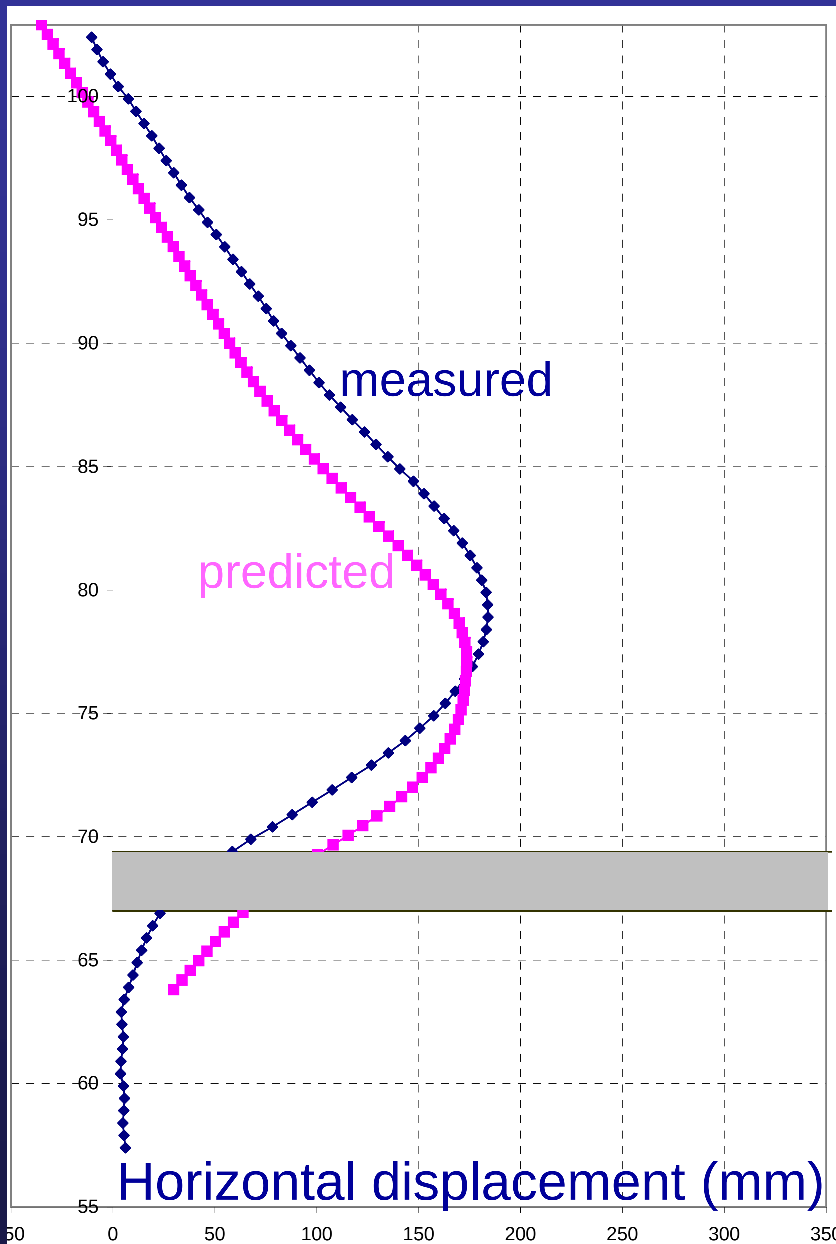


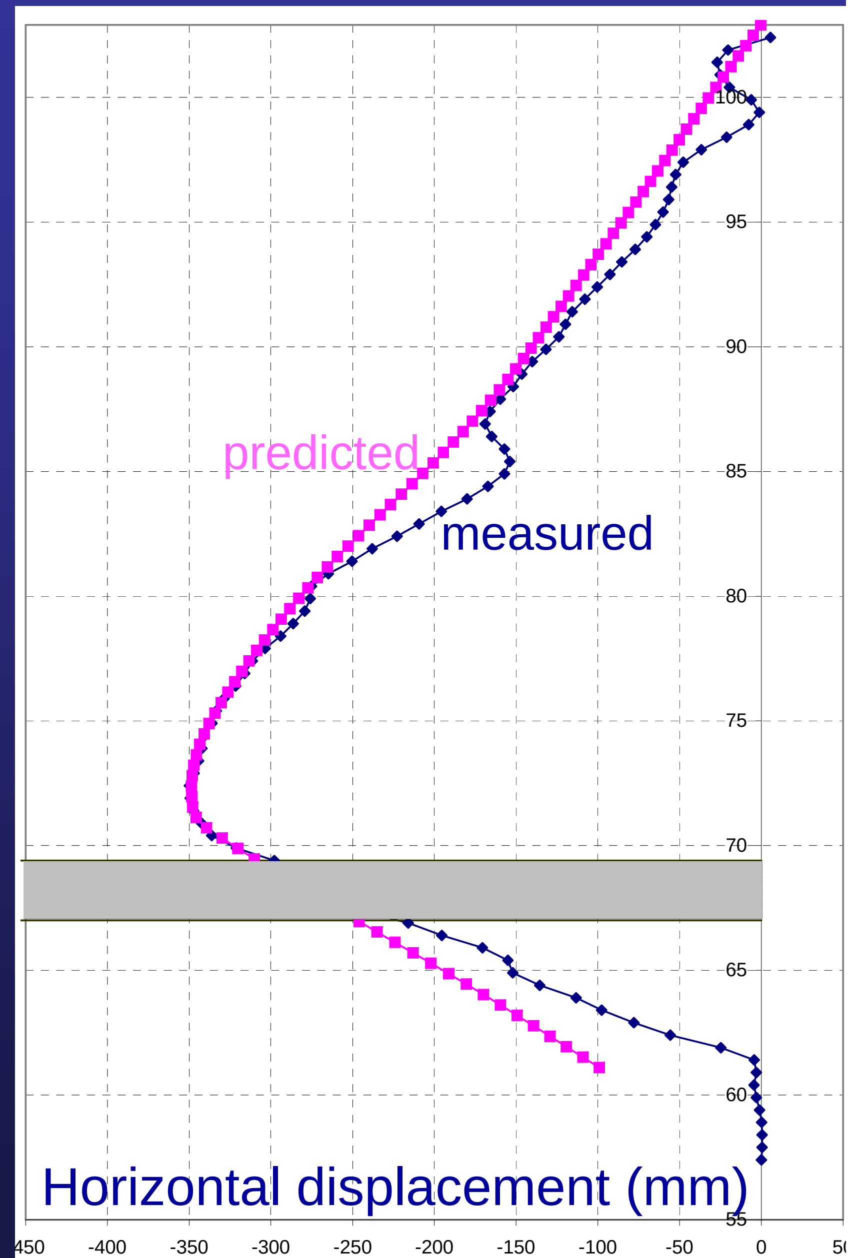


North wall
9th
South wall
Elevation (m RL)
measured
predicted
Horizontal displacement (mm)
100
95
90
85
80
75
70
65
60
55
50
0
50
100
150
200
250
300
350
predicted
measured
Horizontal displacement (mm)
450
-400
-350
-300
-250
-200
-150
-100
-50
0
50

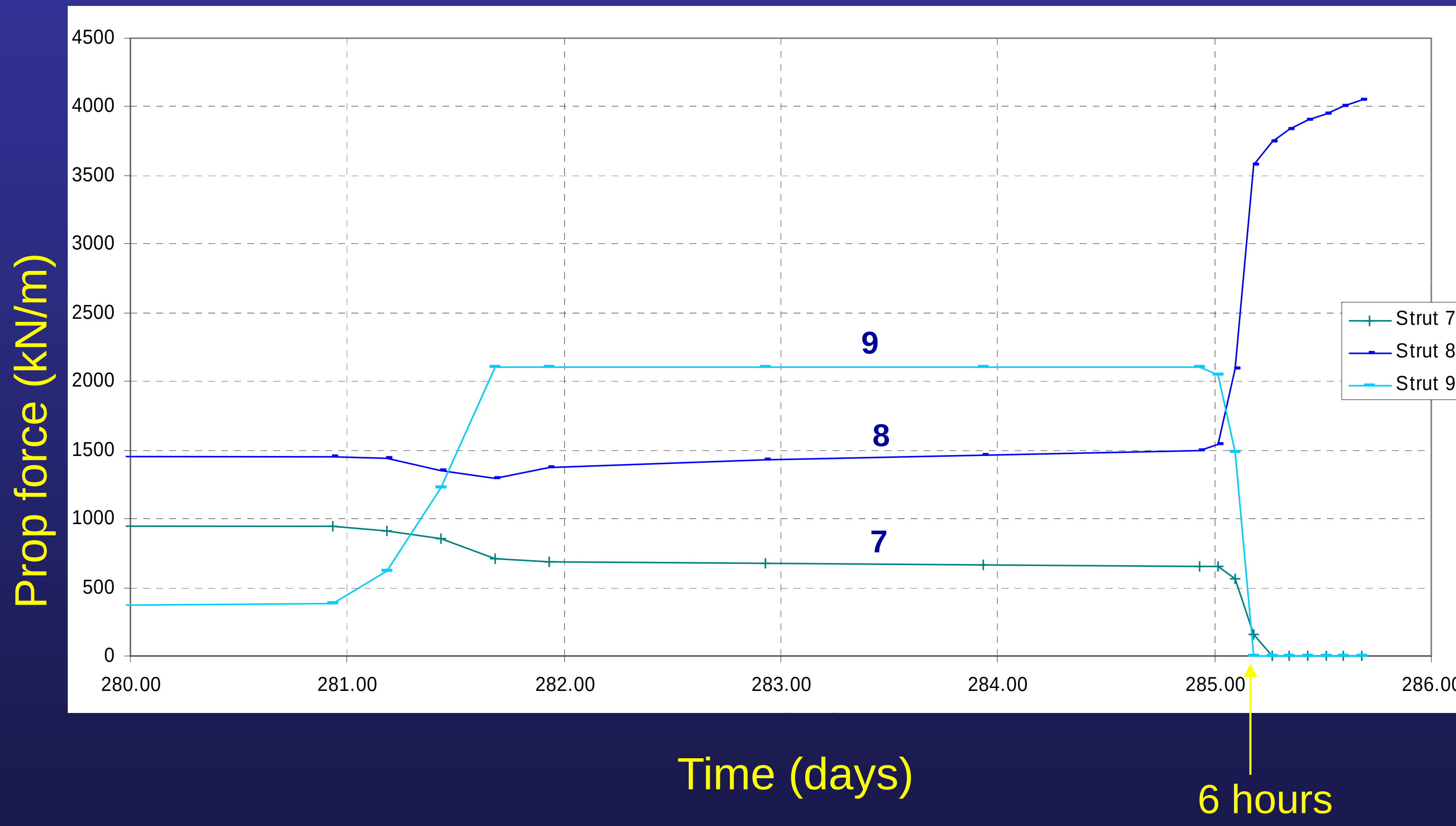
North wall *10th* *South wall*

Elevation (m RL)

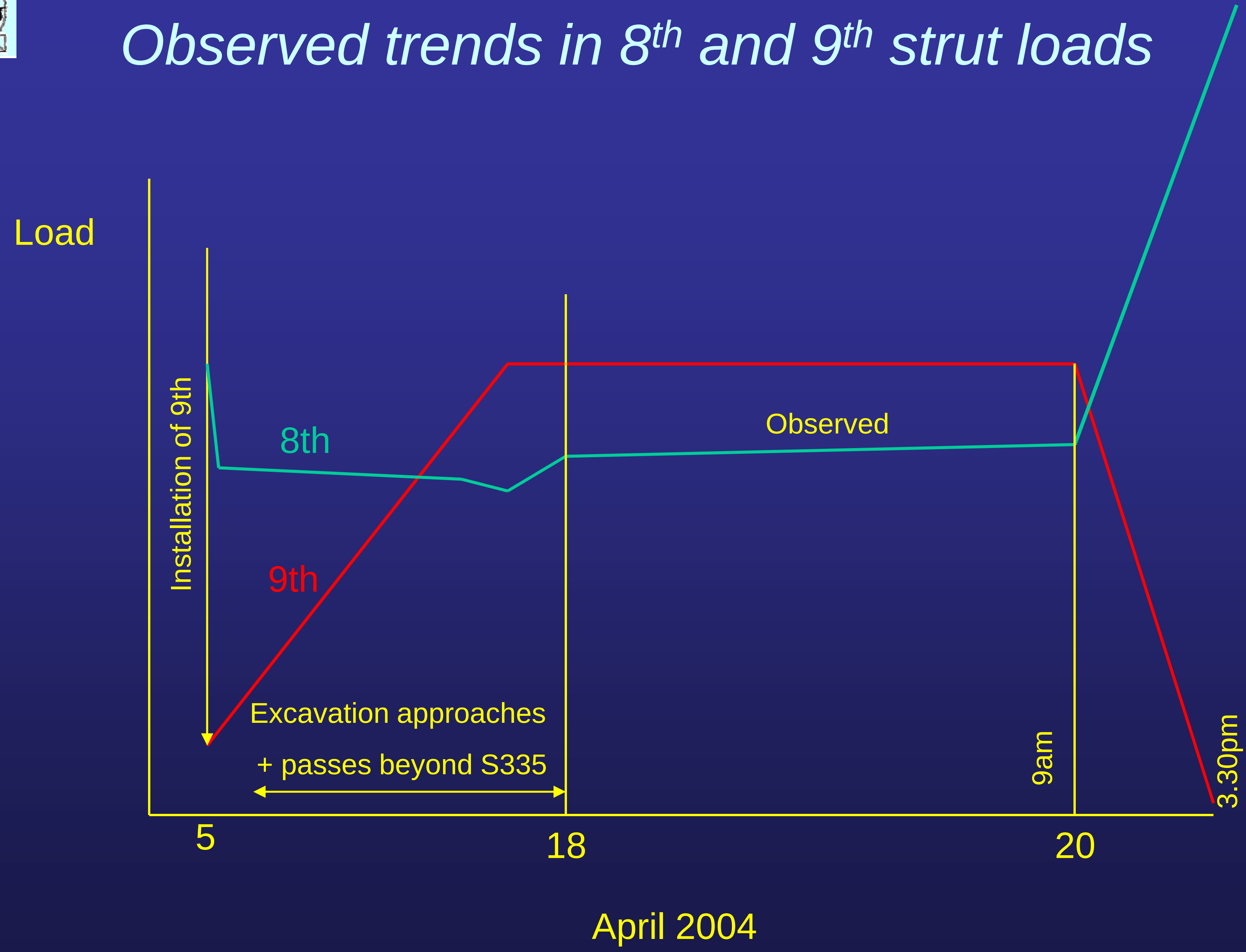




Predicted trends in 7th, 8th and 9th strut loads



Observed trends in 8th and 9th strut loads



- The analyses matched reasonably well the build up in horizontal wall movements, and the trends in the forces in the 7th, 8th and 9th level struts

- To match movements and forces at all stages it was necessary to model the jet grout as a brittle material

- The upper JGP was predicted to pass its peak strength during excavation to the 6th level and the lower JGP to pass its peak strength during excavation to the 9th level

- The 9th level strut reached its capacity during excavation to the 10th level

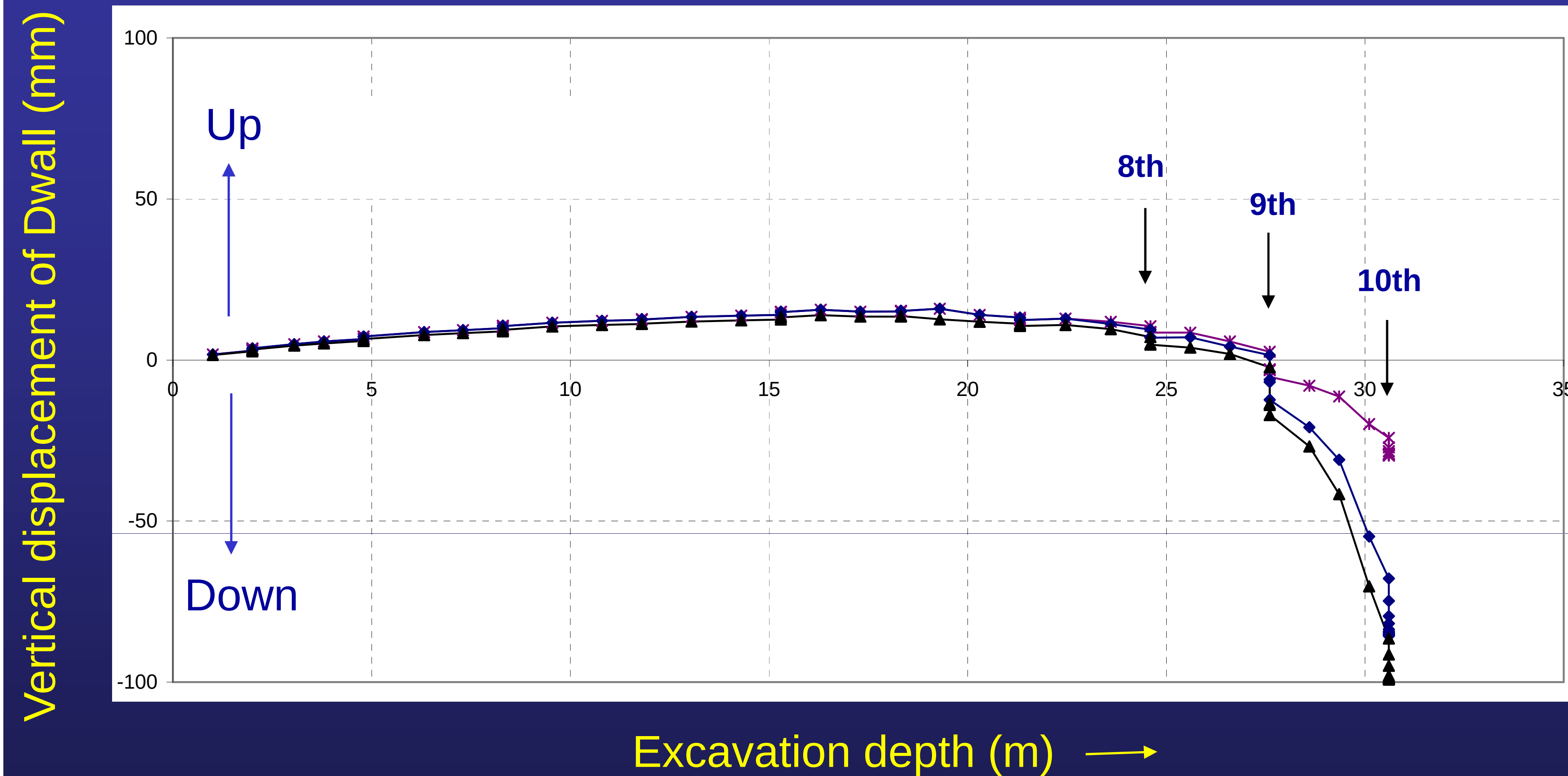
Key findings from geotechnical analyses

- The collapse had to be initiated by allowing the 9th level strut to strain soften – a ductile failure of the connection was not associated with a collapse
- The bending moment capacity of the south wall was reached on the first stage of excavation below the 9th level, but a hinge did not form in the wall until the sacrificial JGP layer had been removed

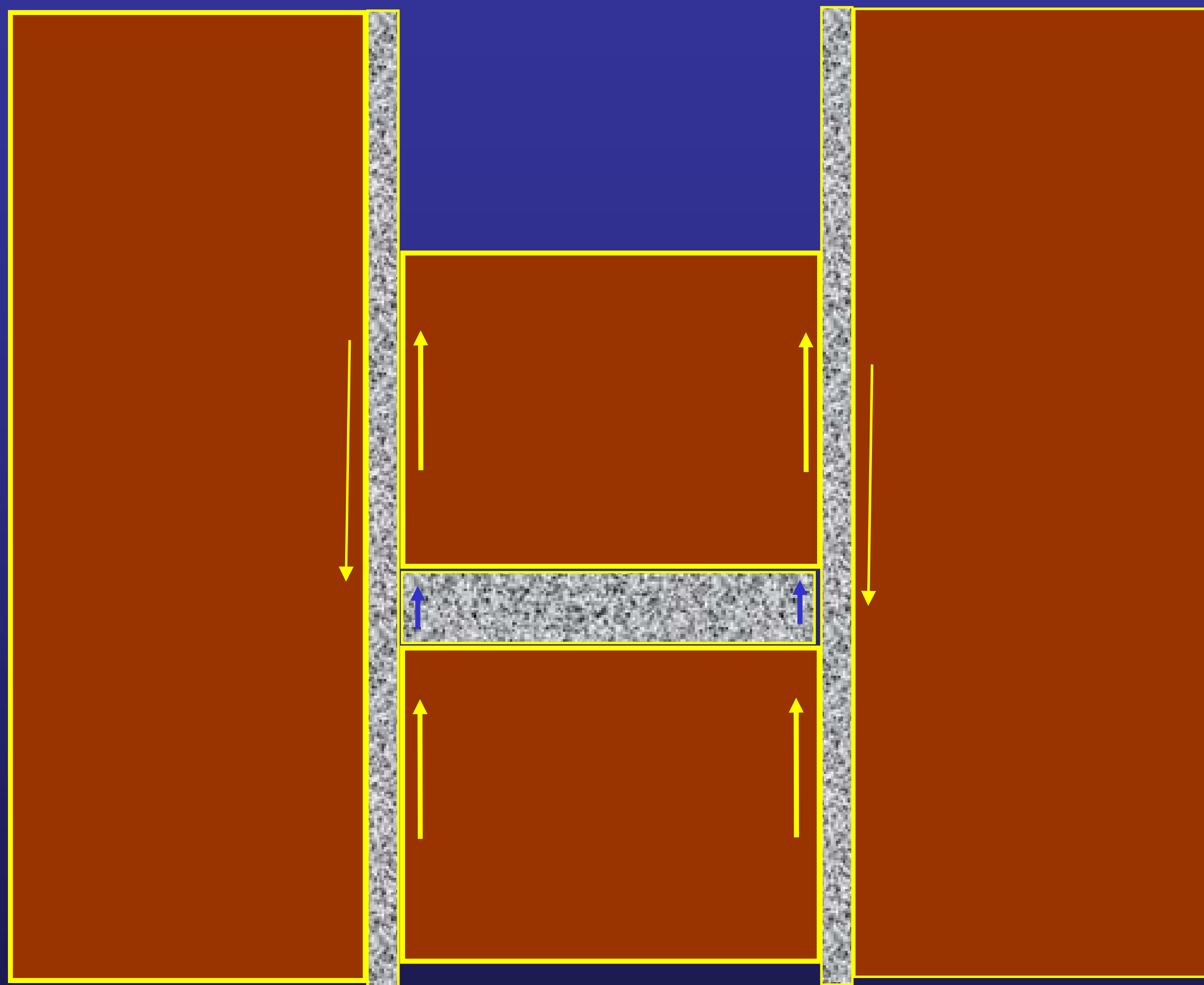
Key findings from geotechnical analyses

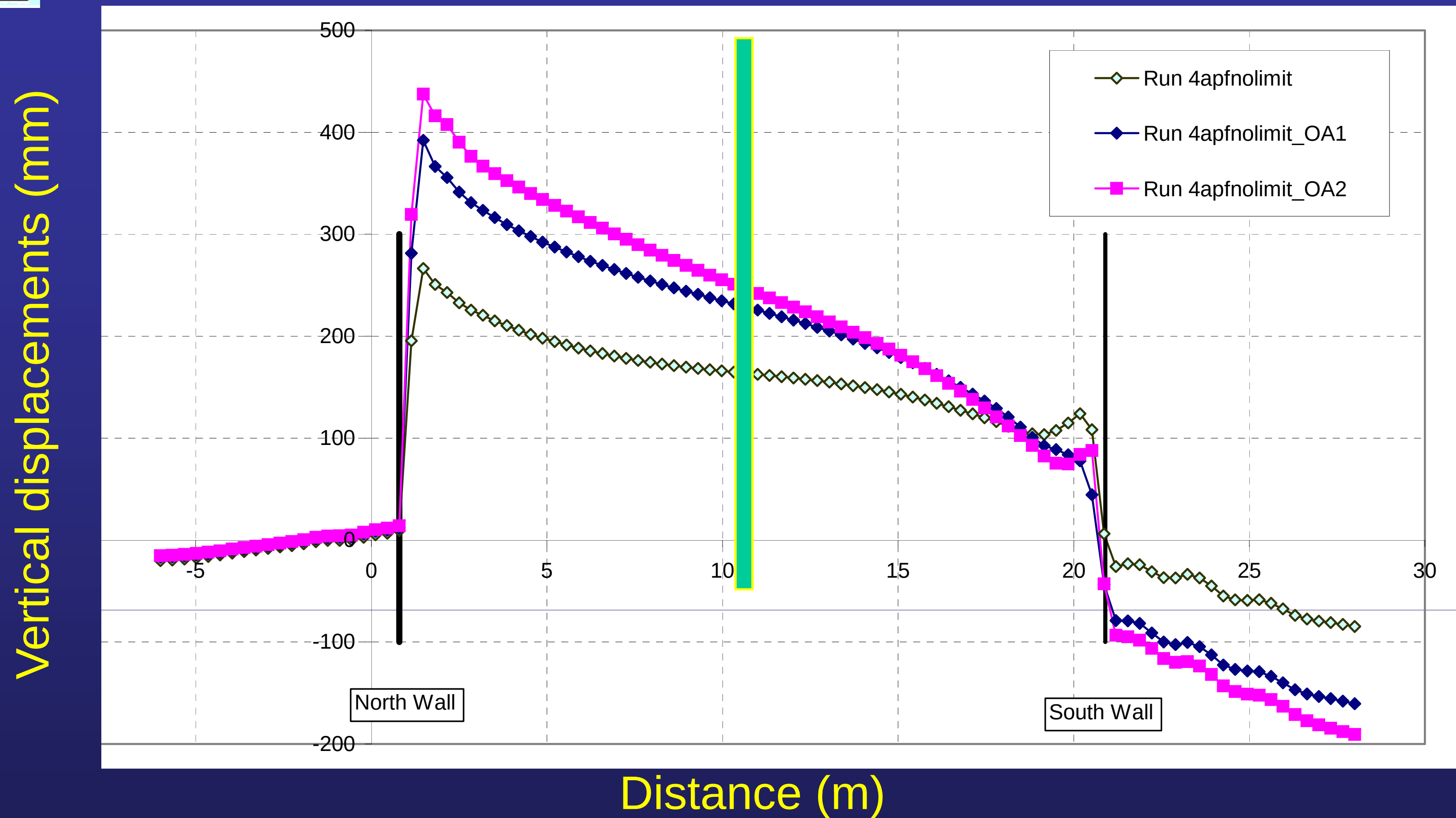
Trigger required to initiate collapse

Relative vertical displacement between the kingposts and Dwall panels



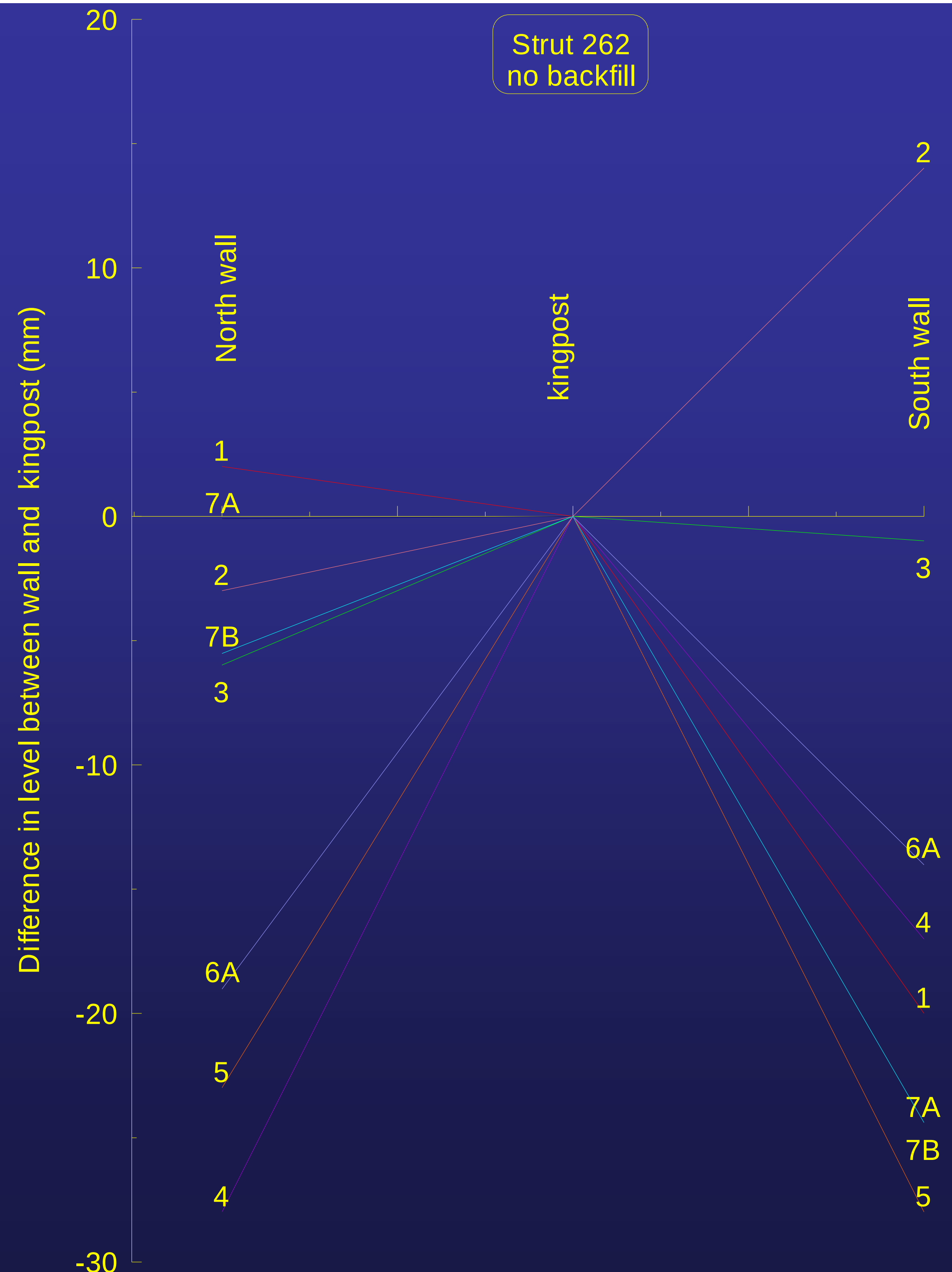
Predicted settlement of south wall during excavation





Predicted vertical displacement of lower JGP layer after excavation to 10th level

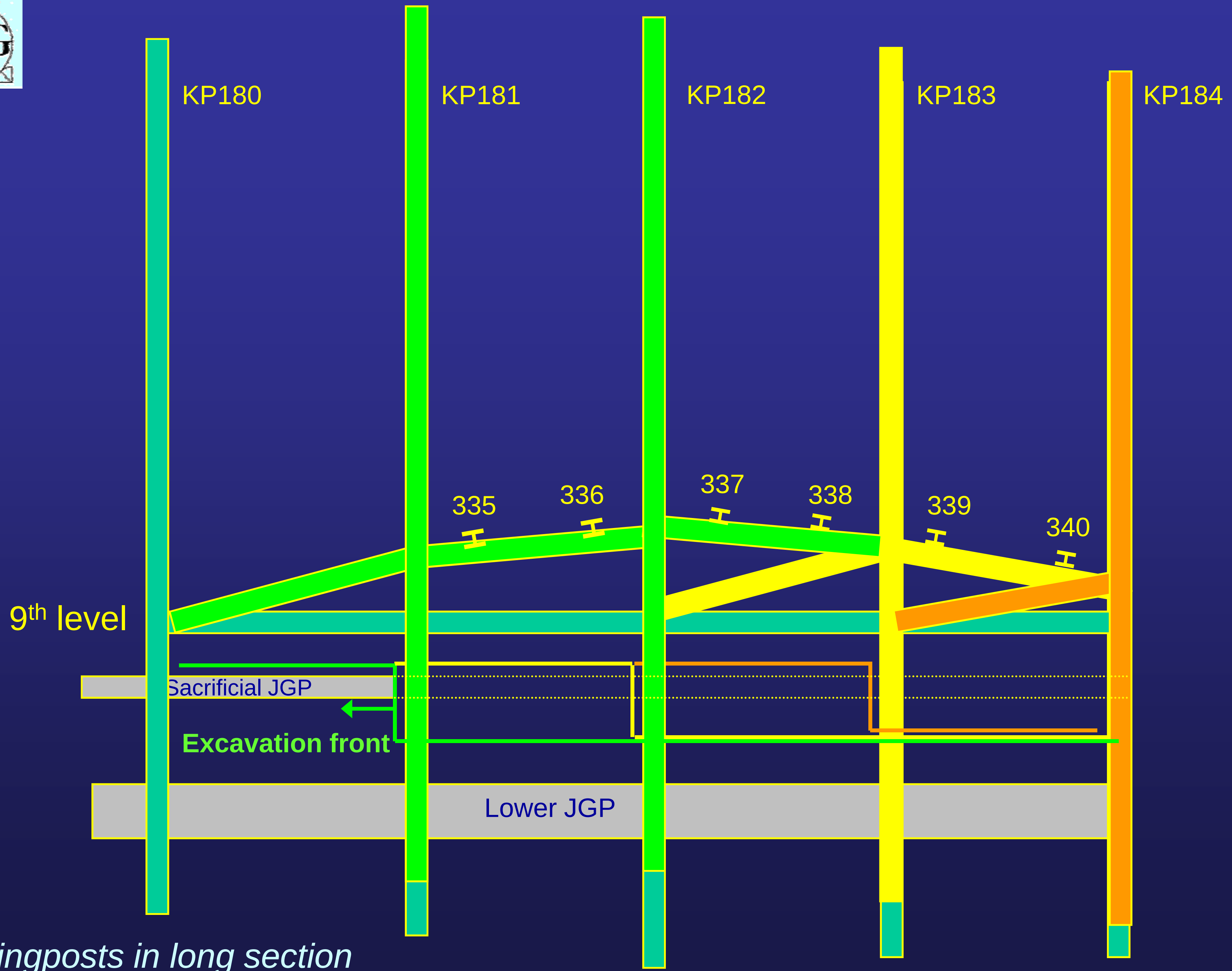
Typical relative displacements of strut between walls and kingpost



- Calibrated FE analyses predict downward displacement of Dwall and upward displacement of kingpost when sacrificial JGP layer excavated

- Survey data supports upward vertical displacement of centre of strut relative to ends

Relative vertical displacement

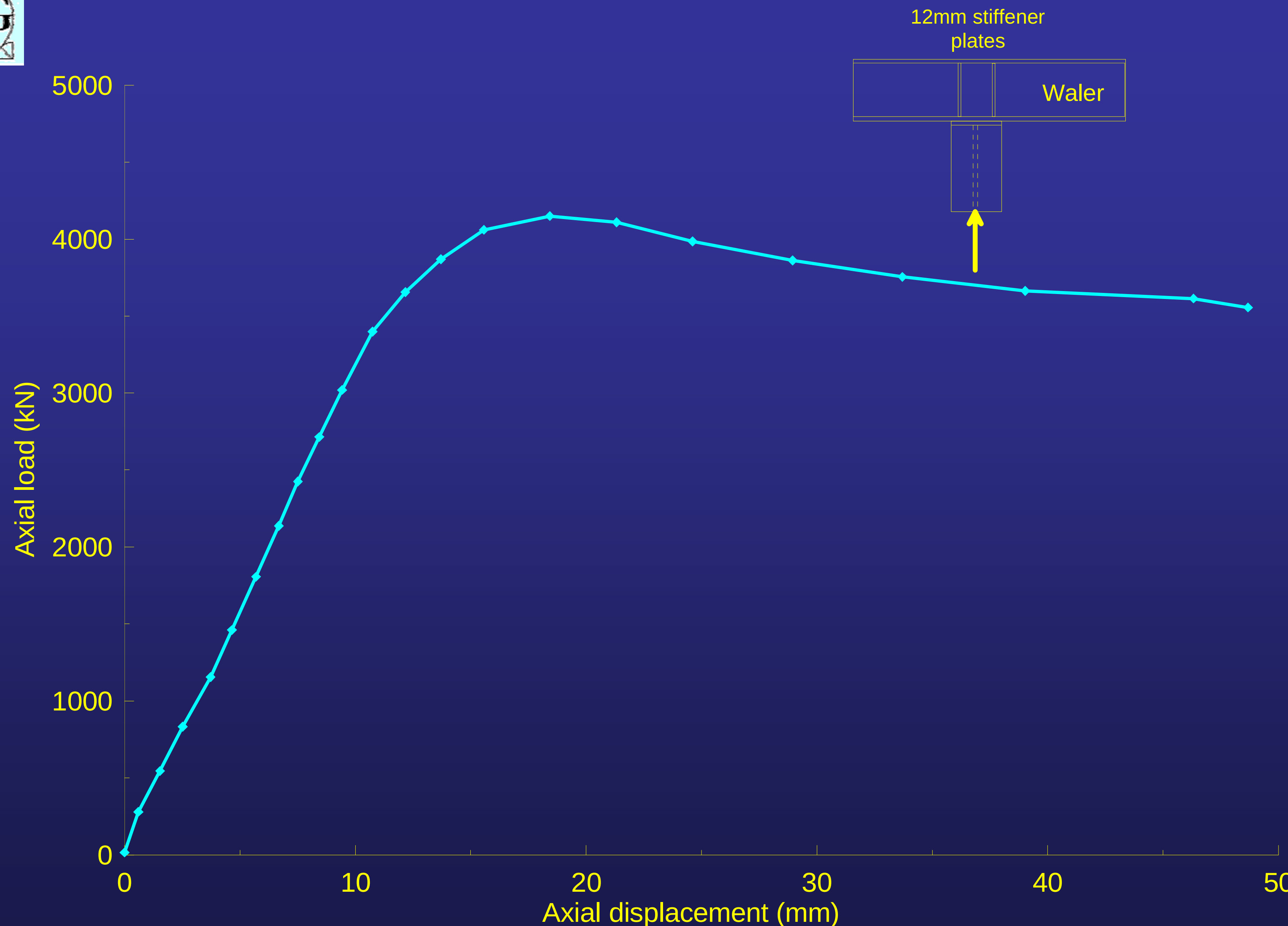


Kingposts in long section

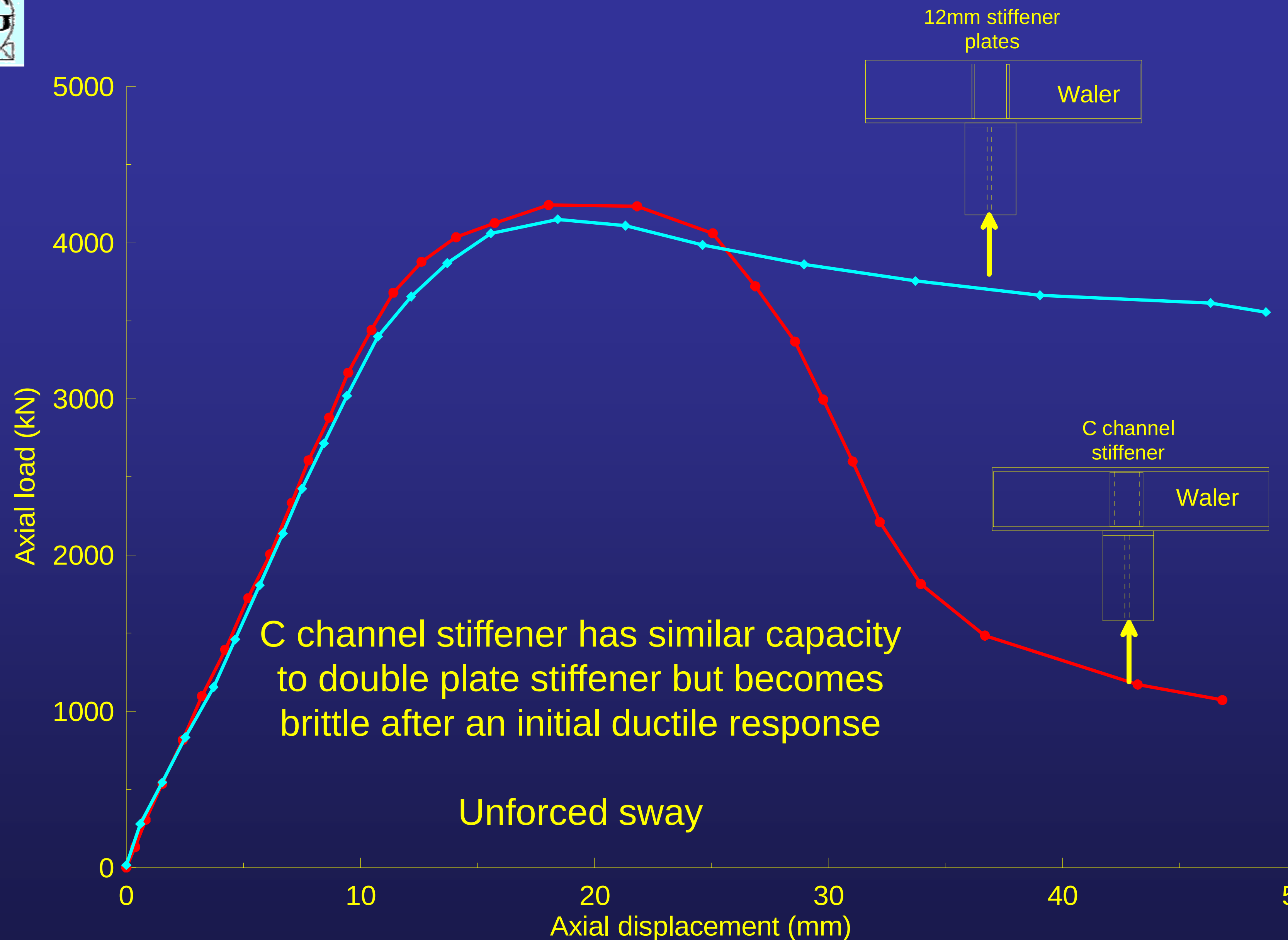
Post-collapse investigations

Structural steel physical tests and numerical analyses.

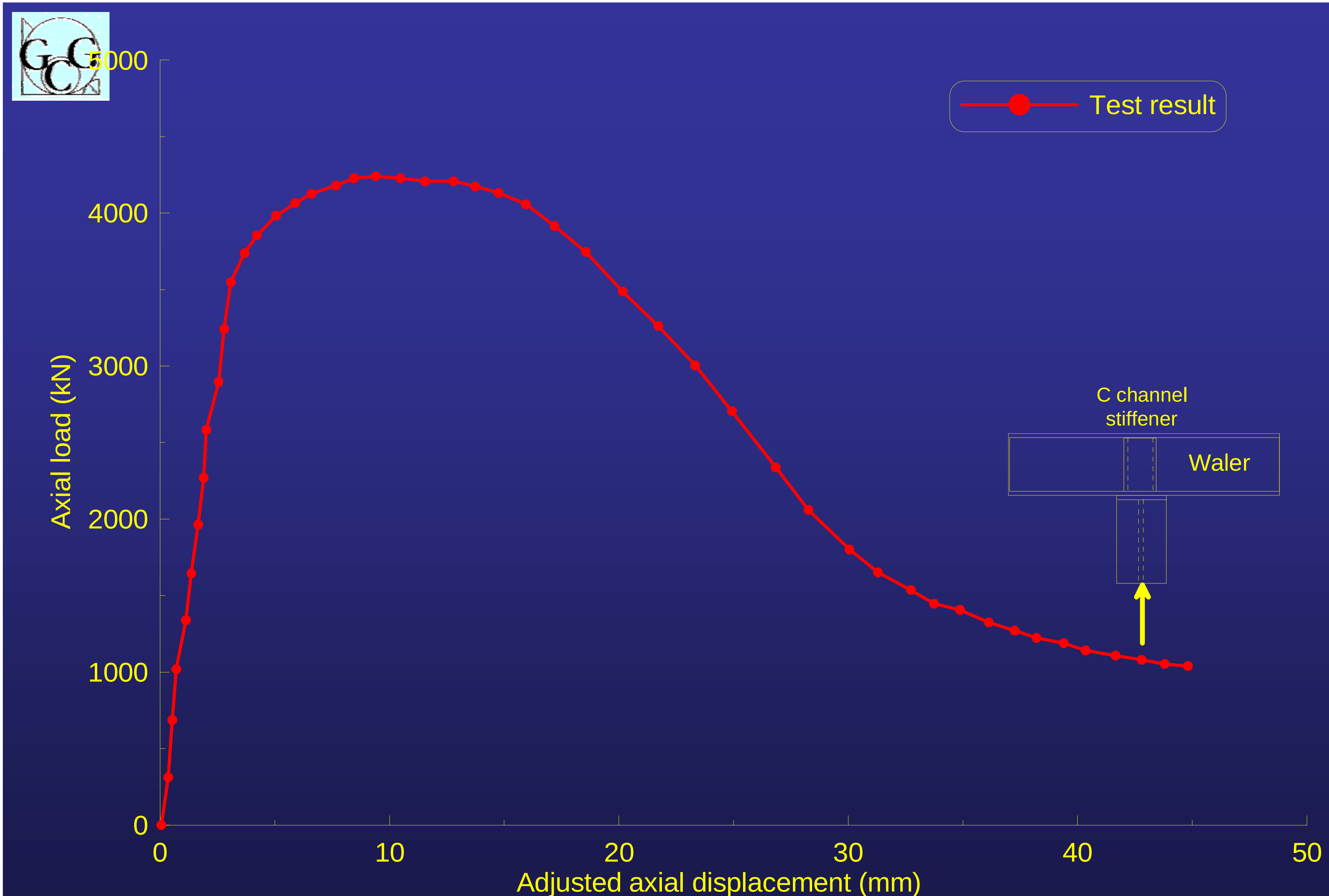
The effect of relative vertical displacement



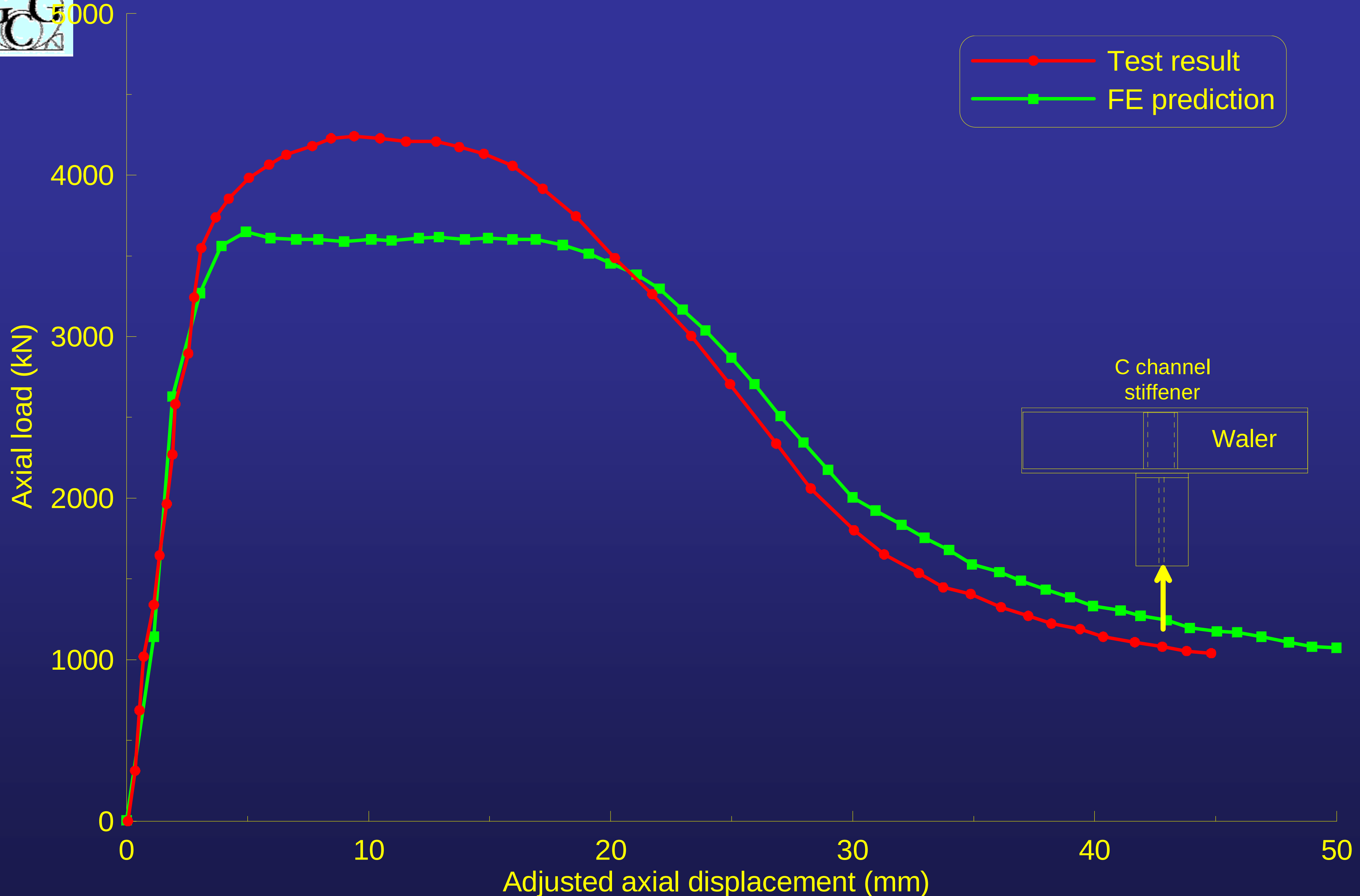
Comparison of tests on connections with plate and C channel stiffeners



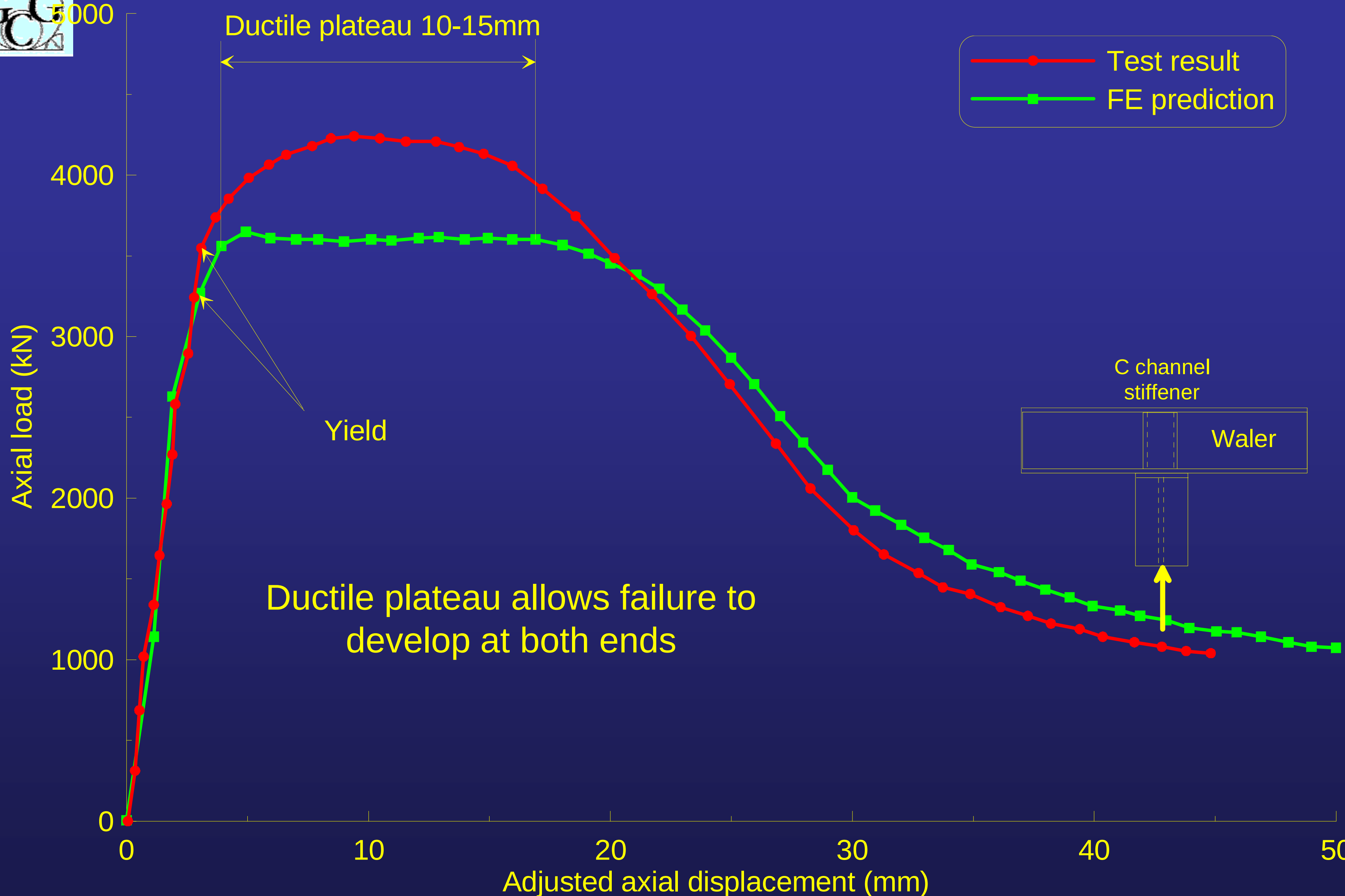
Comparison of tests on connections with plate and C channel stiffeners



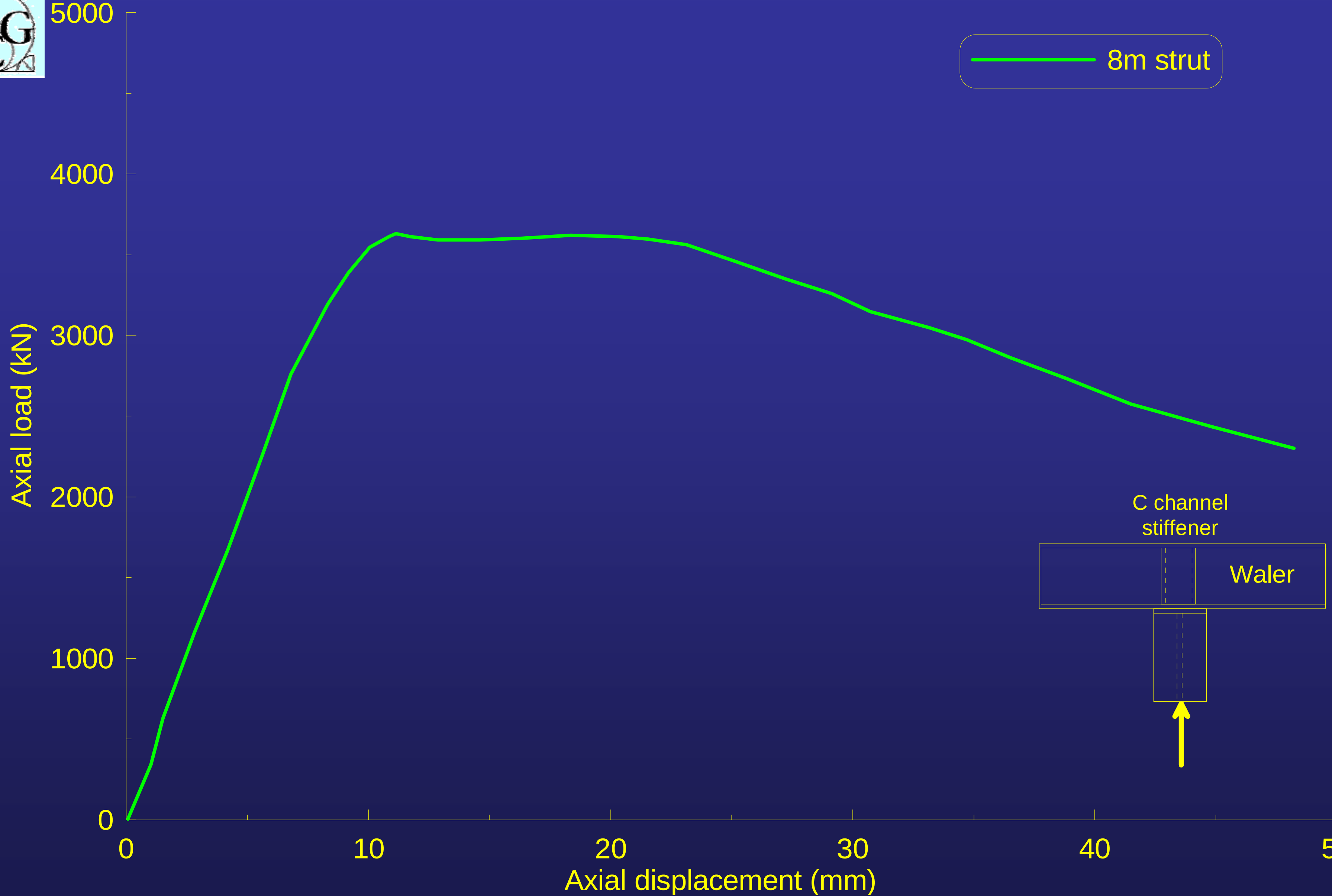
Calibration of FE model of strut-waler connection



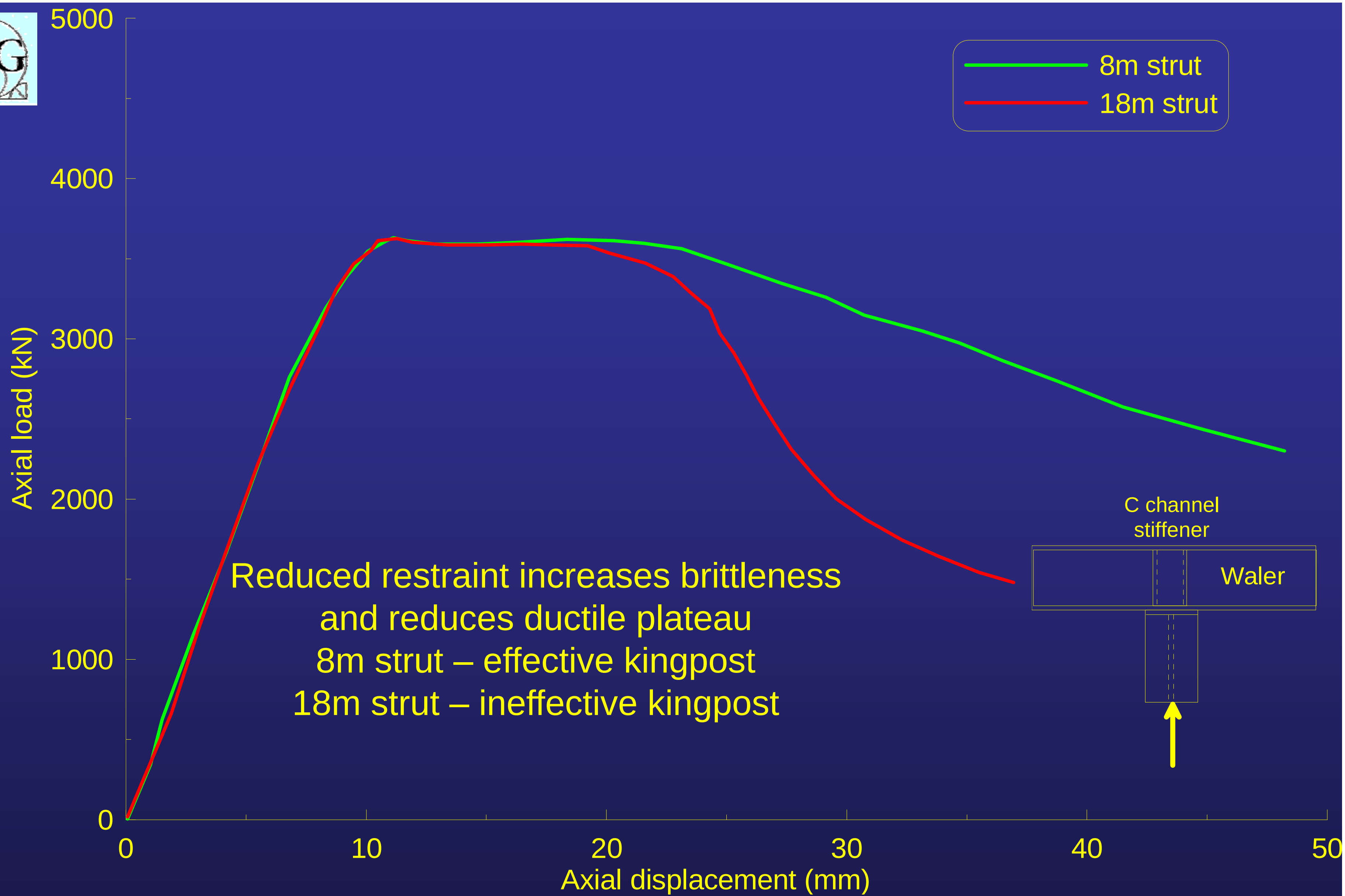
Calibration of FE model of strut-waler connection



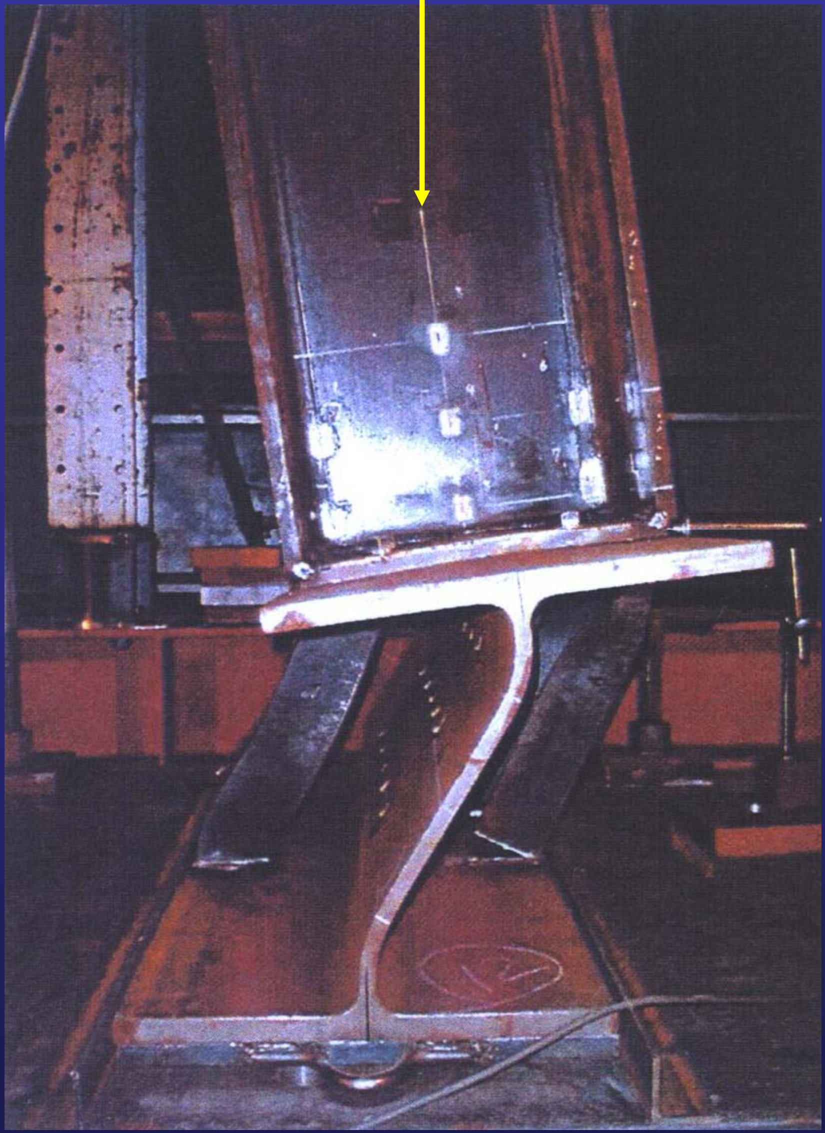
Calibration of FE model of strut-waler connection



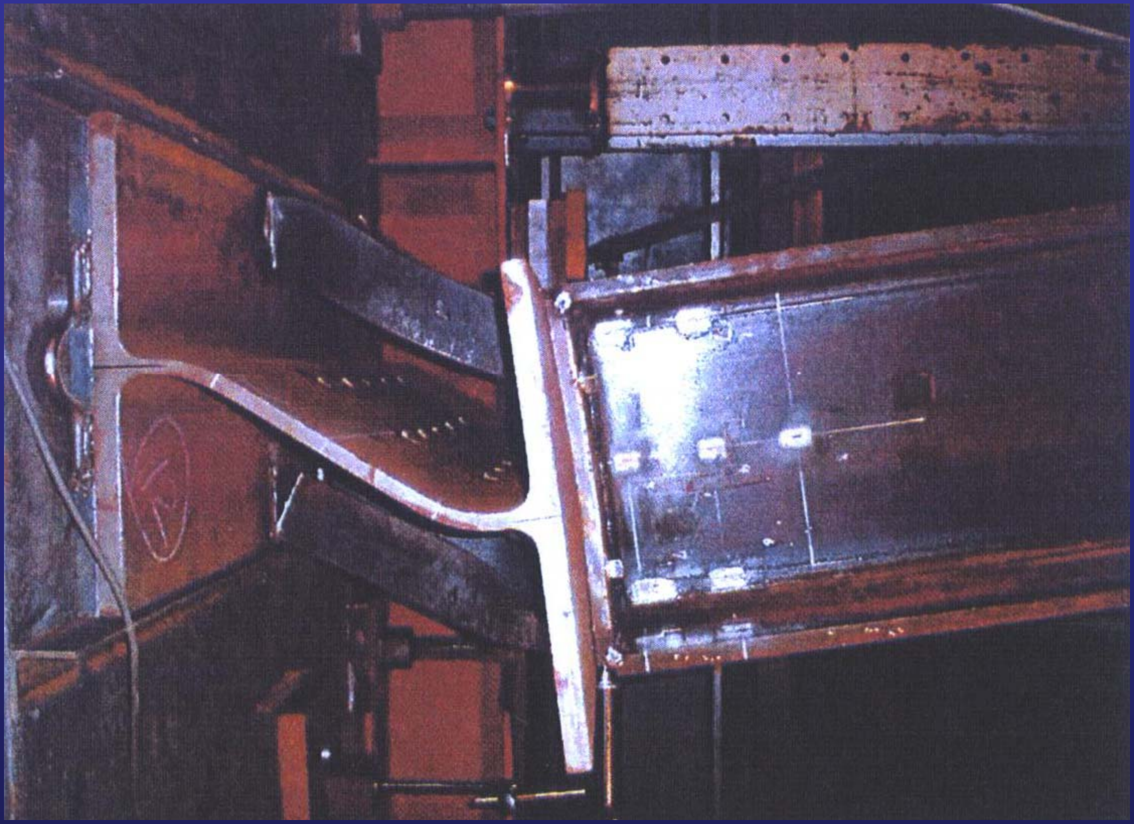
Effect of strut length (bending restraint) on brittleness of connection



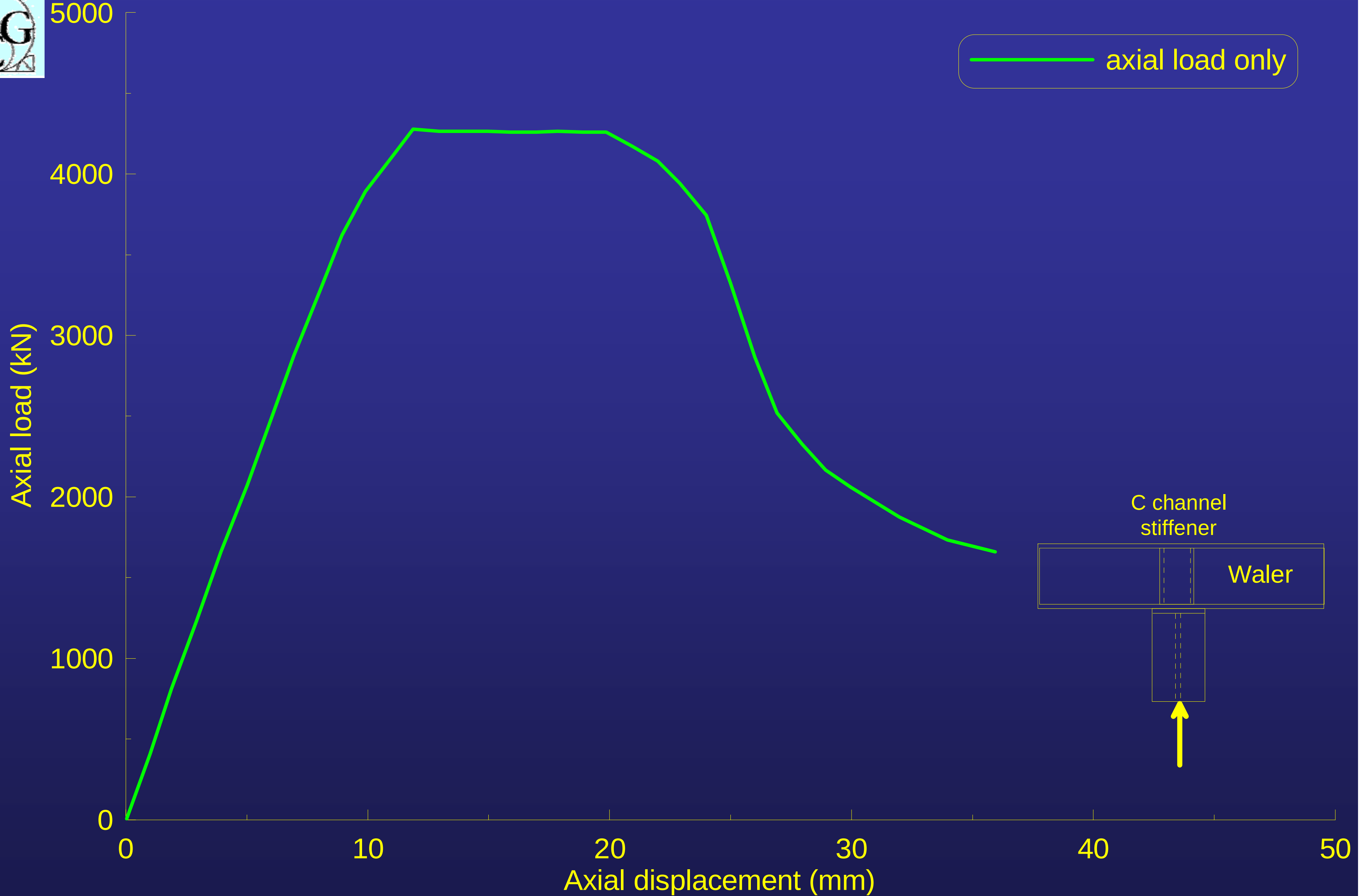
Effect of strut length (bending restraint) on brittleness of connection



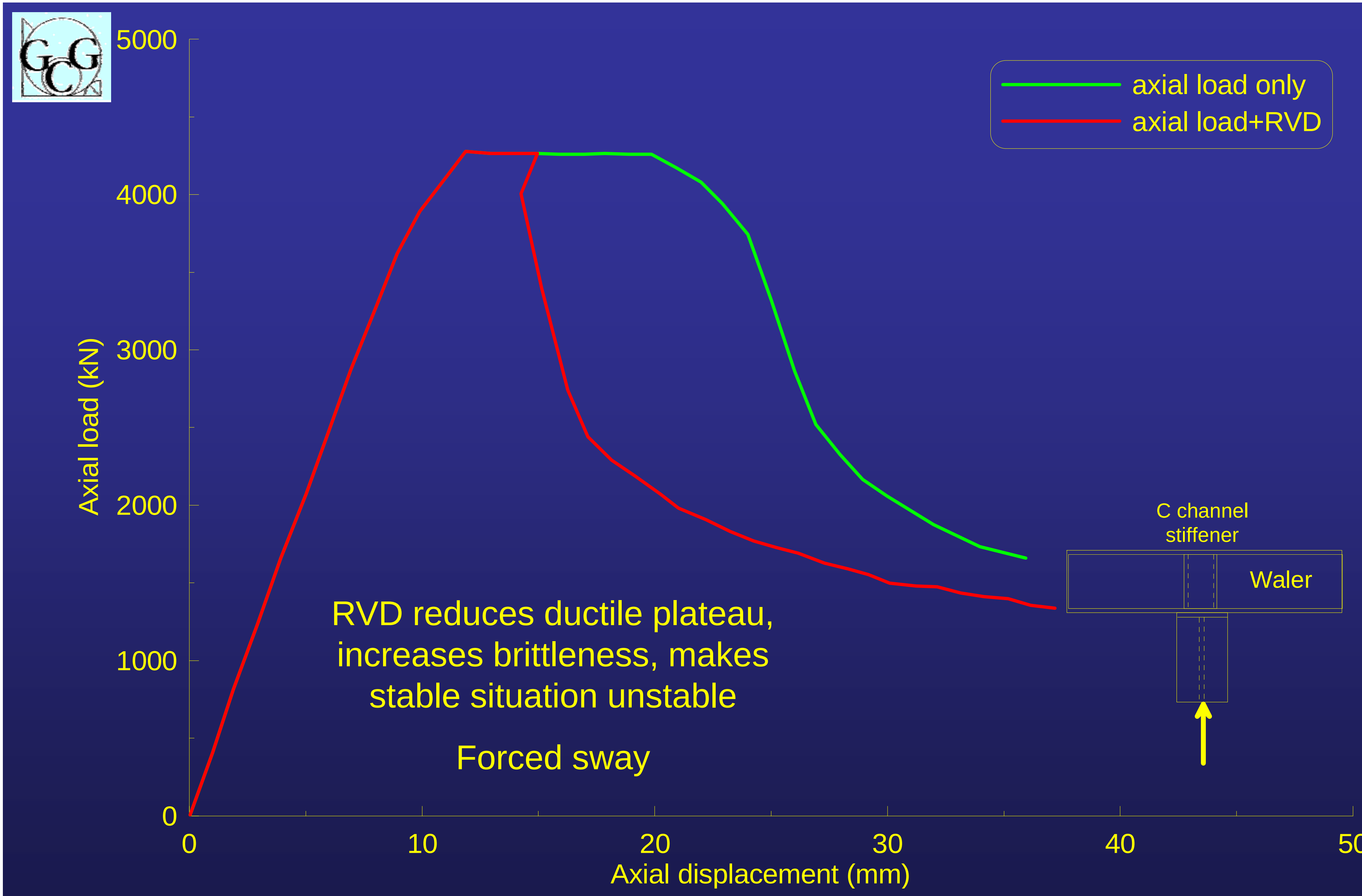
Test on C channel stiffened connection by Nishimatsu



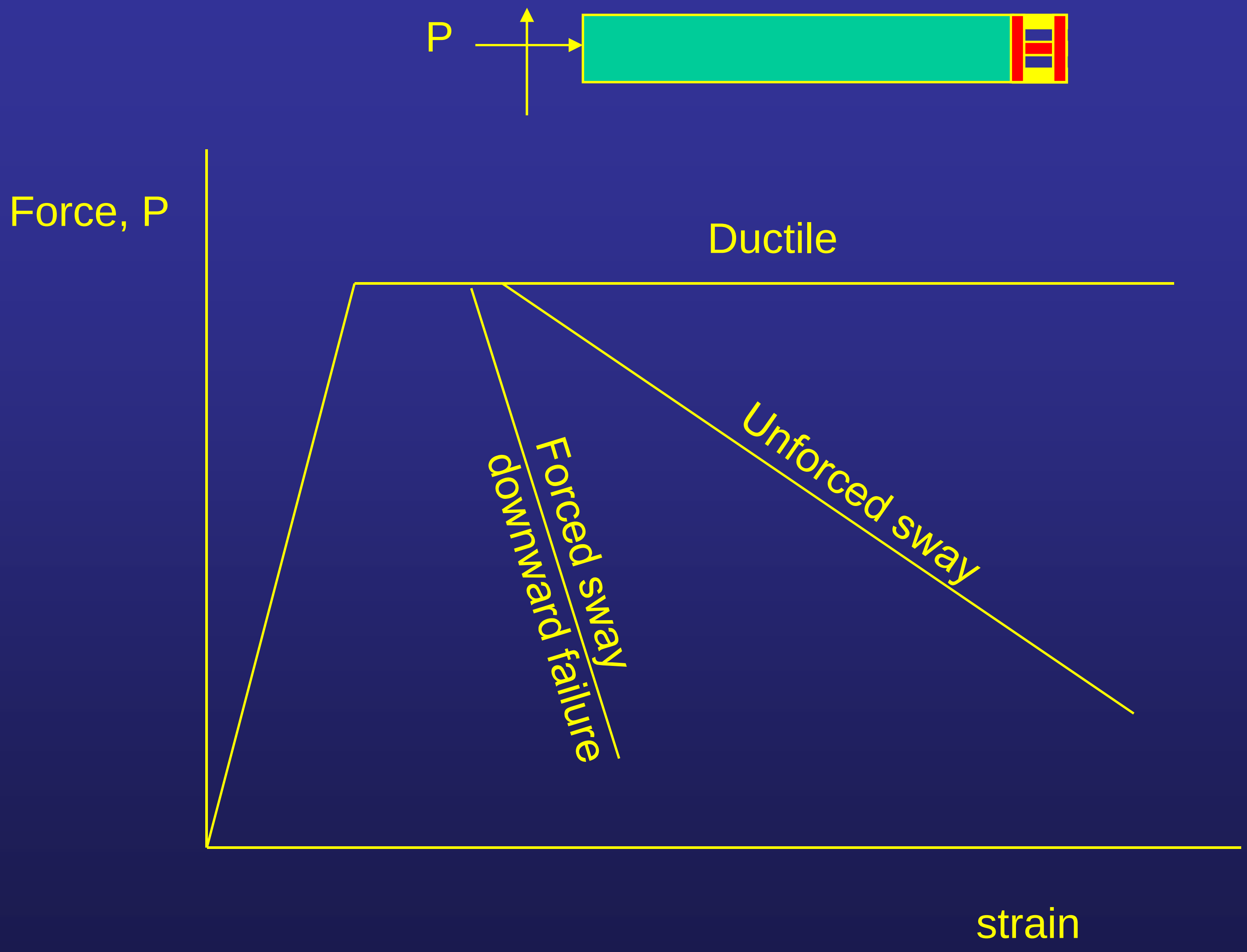
Test on C channel stiffened connection by Nishimatsu



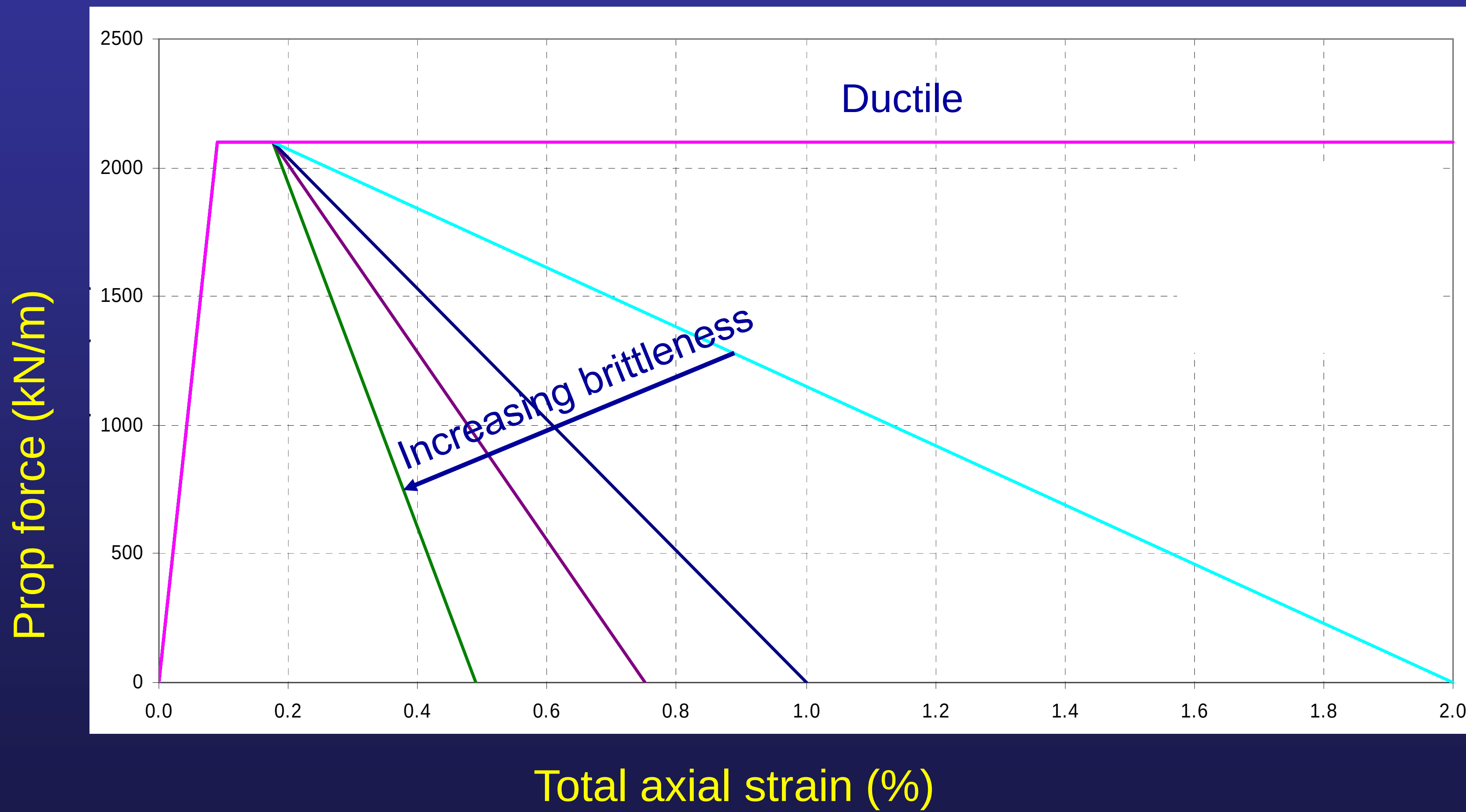
Effect of relative vertical displacement



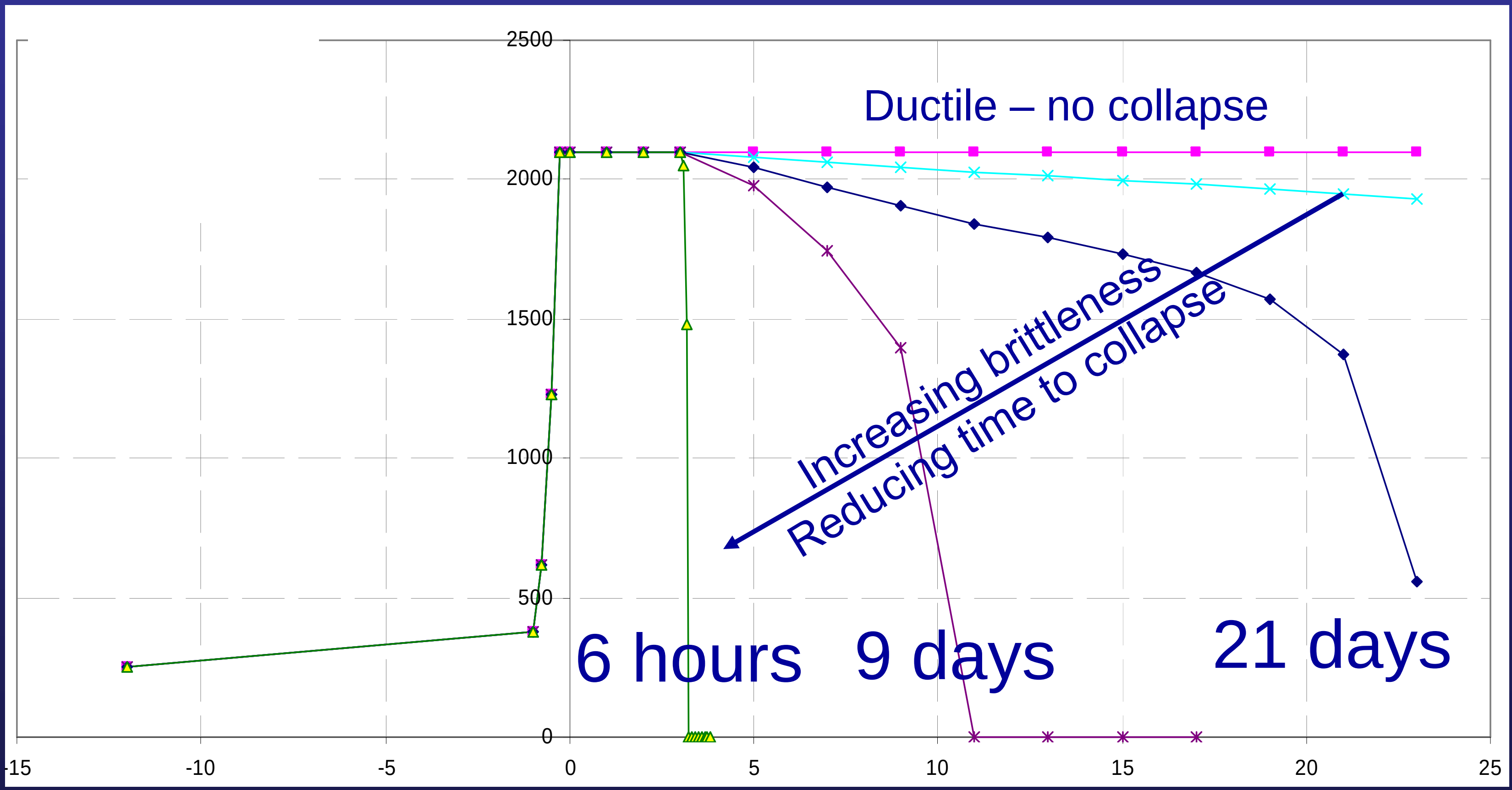
Effect of relative vertical displacement


P
Force, P
Ductile
Forced sway
downward failure
Unforced sway
strain

Effect of brittleness of strut to waler connection

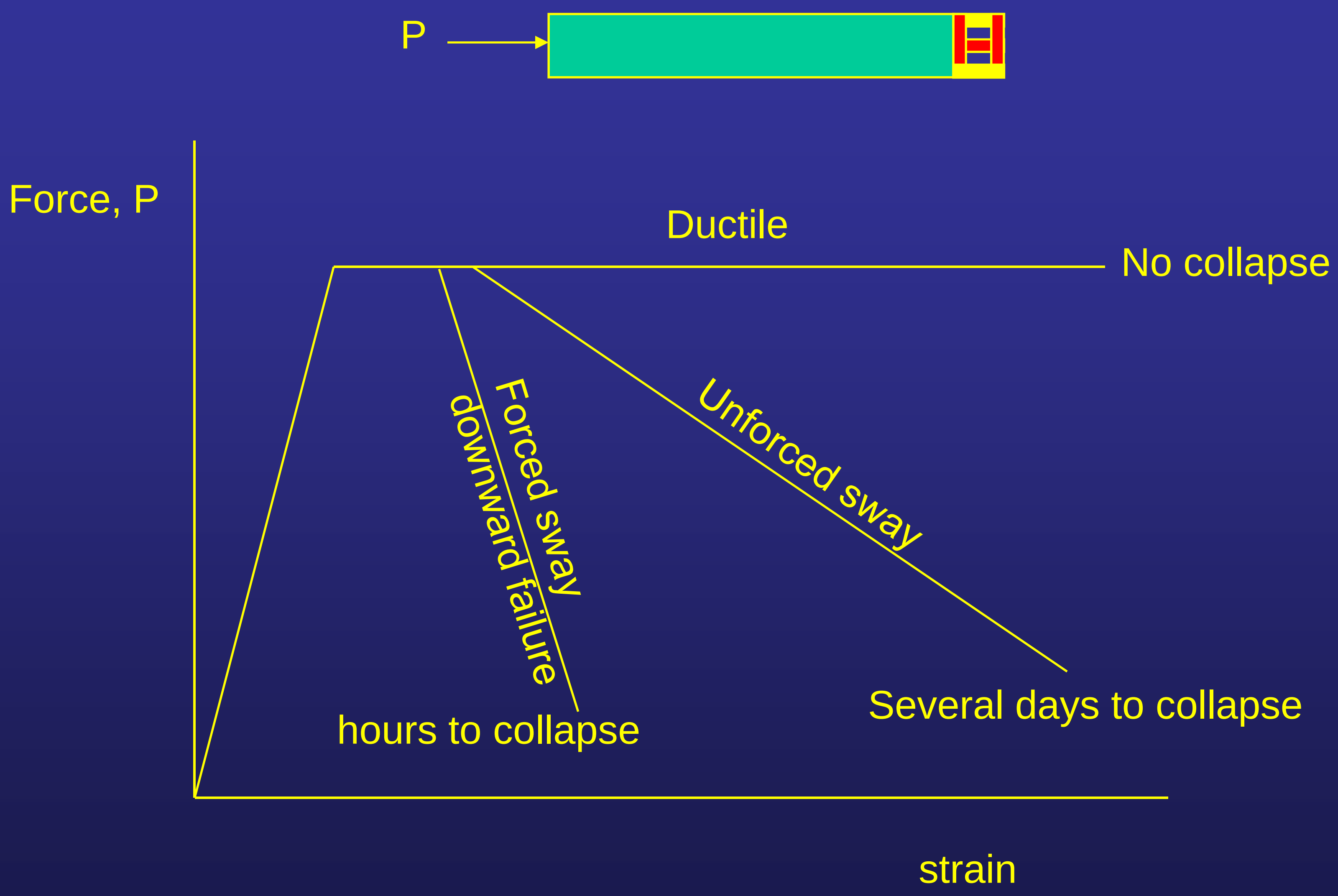


Effect of brittleness of strut to waler connection



Prop force (kN/m)

Time since reaching excavation level of 72.3m RL (days)


P
Force, P
Ductile
No collapse
Forced sway
downward failure
Unforced sway
hours to collapse
Several days to collapse
strain

- There was relative vertical displacement between the diaphragm wall, which settled when the sacrificial JGP was removed, and the kingpost, which rose, i.e. relative vertical displacement between the ends and centre of the strut (RVD)

- The strut-waler connection was ductile-brittle. The ductile plateau explains why both ends could fail

- The brittleness of the connection determines the time taken for the collapse to develop

Trigger required to initiate collapse

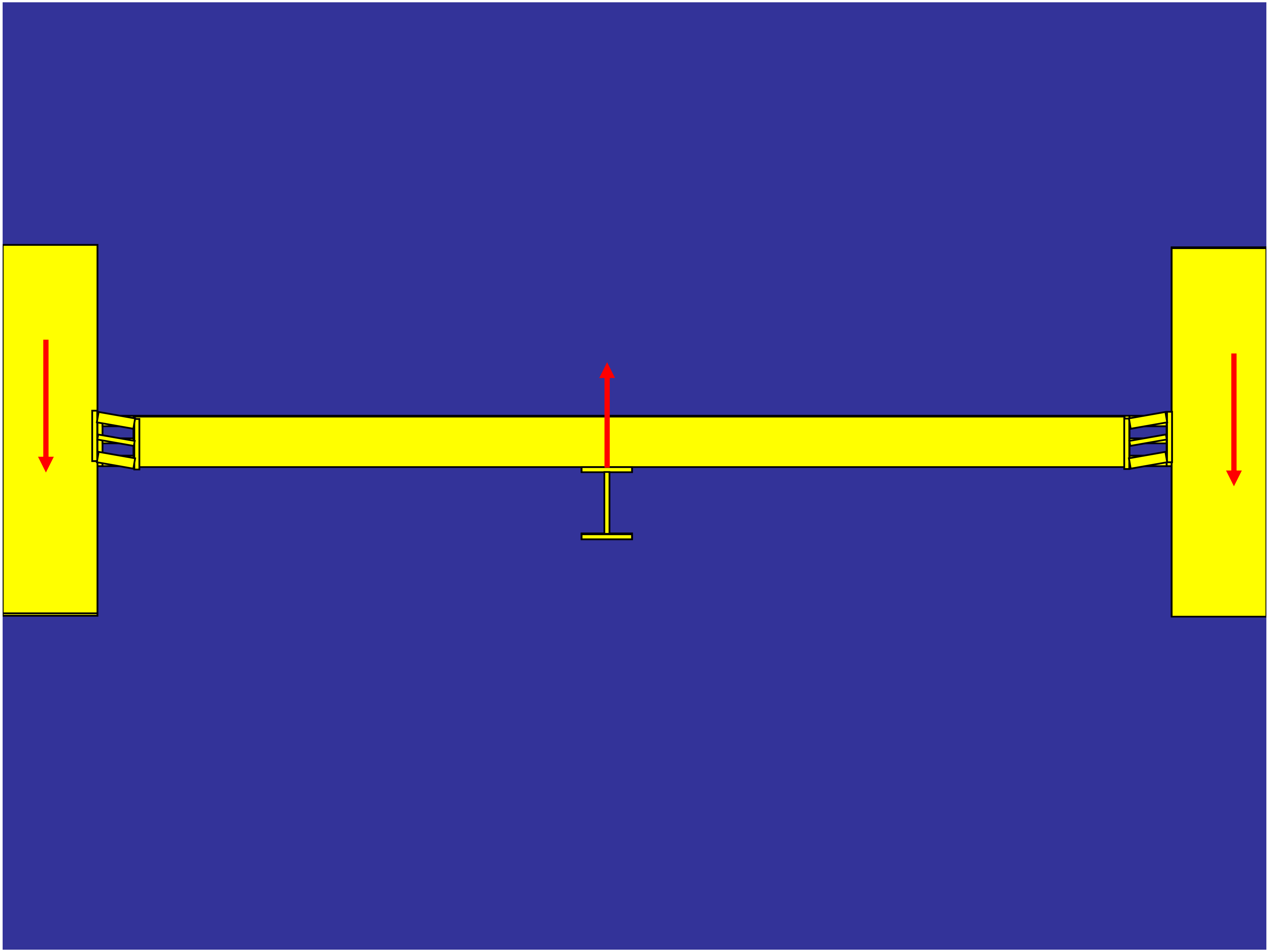
- RVD reduces the length of the ductile plateau and increases the brittleness
- RVD can make a stable situation unstable
- RVD can shorten the time to collapse

- Why downward failure at both ends?
- Why collapse after crossing the buried valley?

Trigger required to initiate collapse

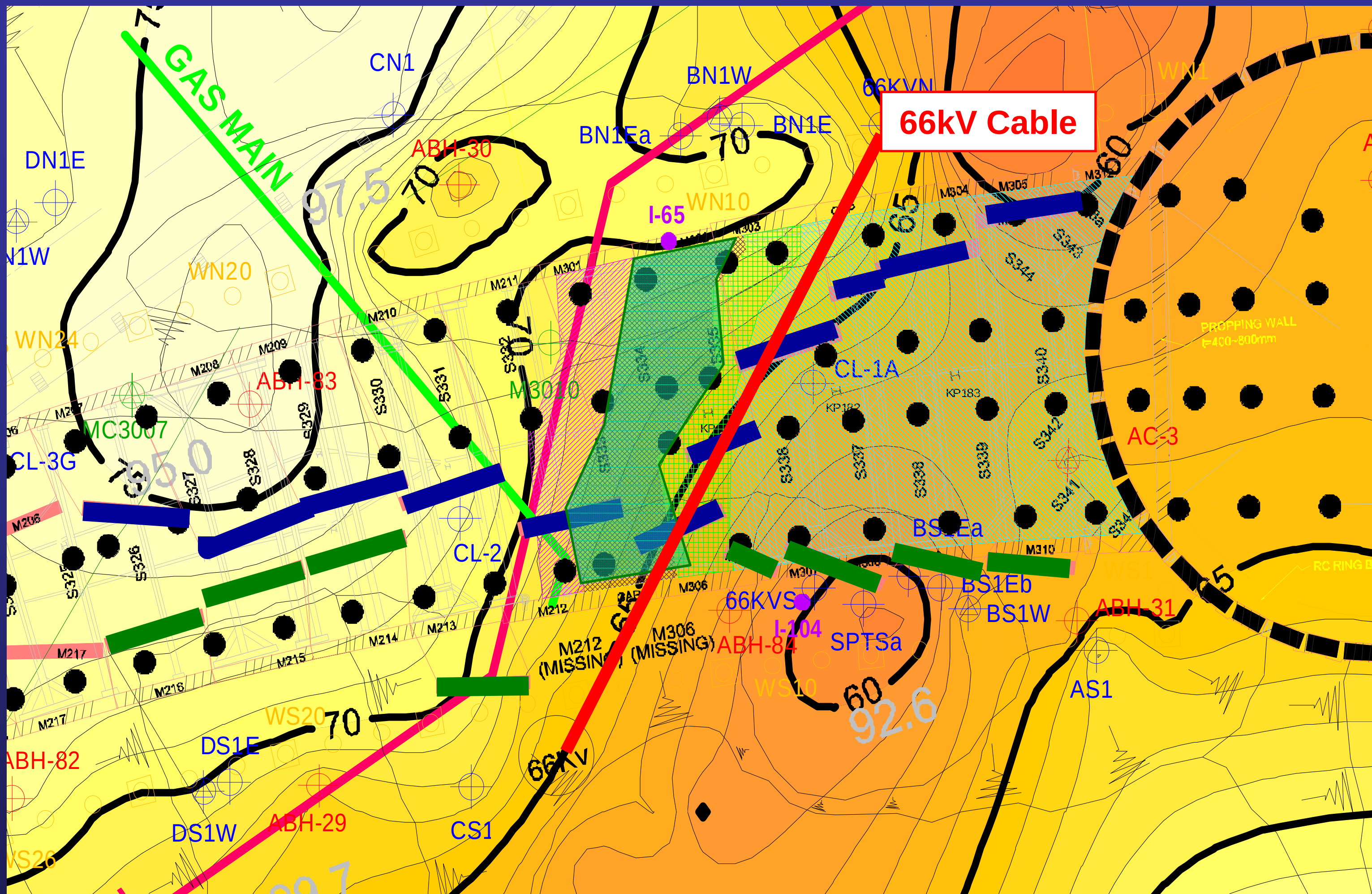
- Free
- Fixed
- Forced sway

Downward failure at both ends

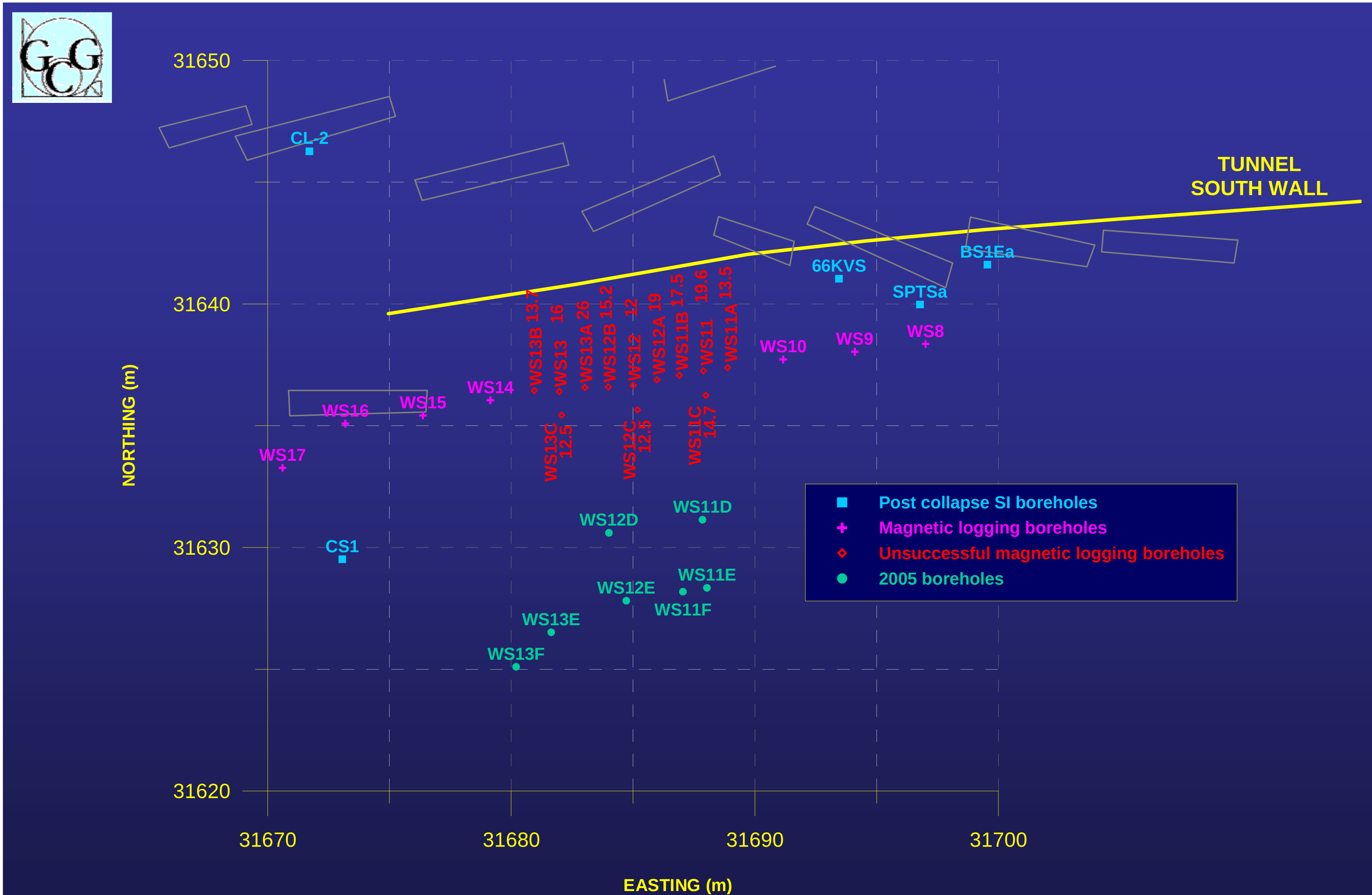


Trigger required to initiate collapse

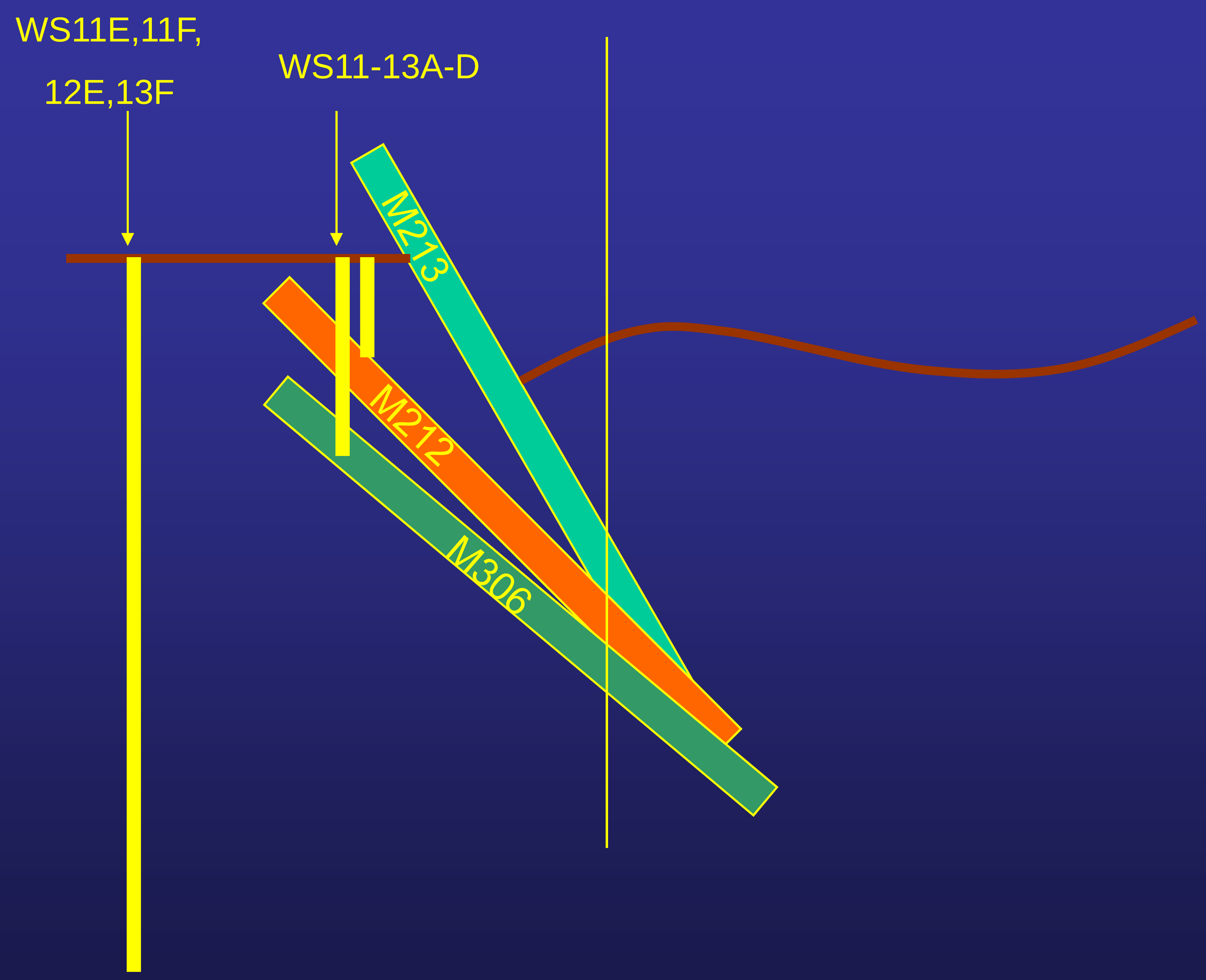
Post-collapse positions of the Dwall panels



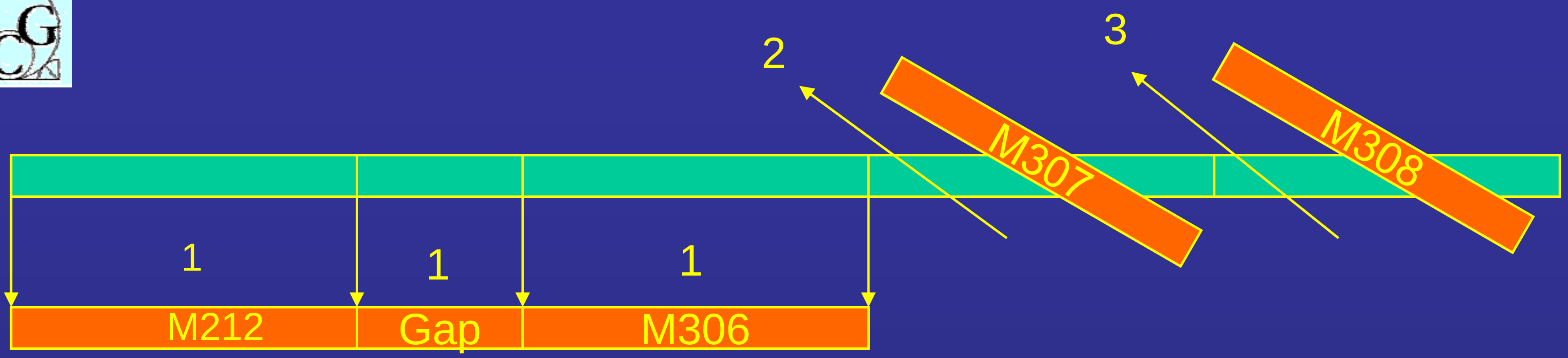
Post-collapse geometry superimposed on buried valley



Coincidence between inability to advance boreholes and missing Dwall panels

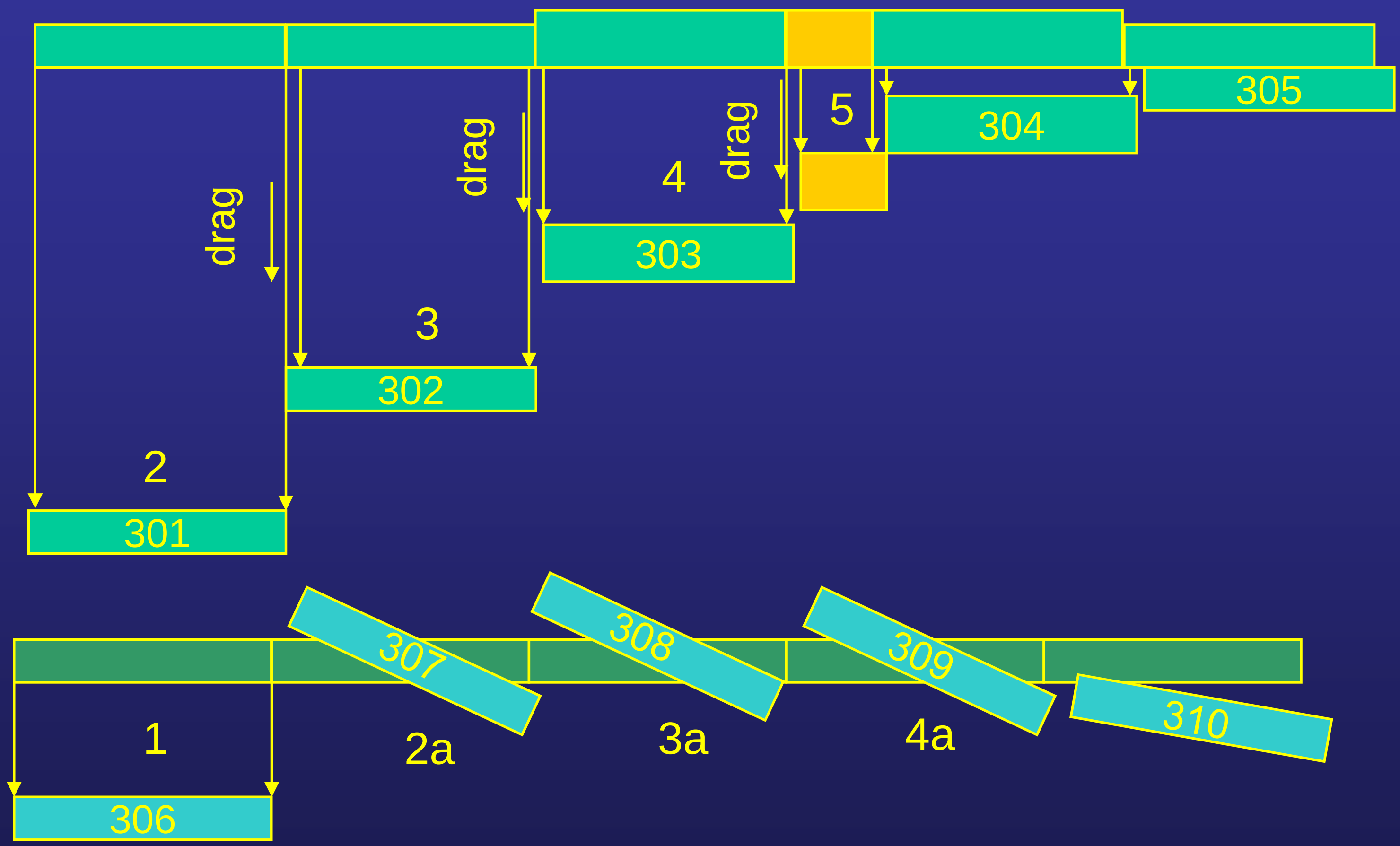


Obstruction created by missing Dwall panels



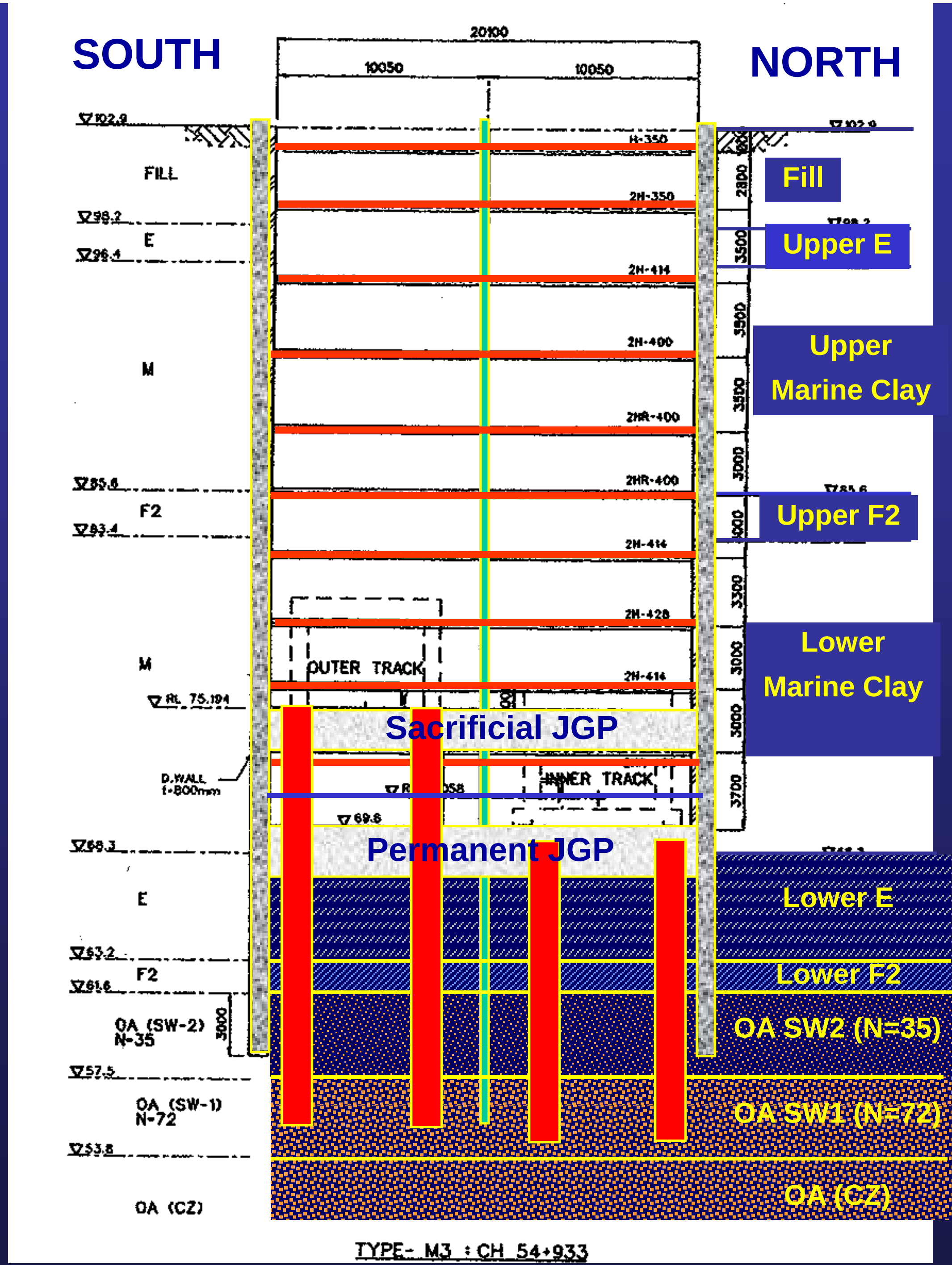
1. Panels M306 and M212, each side of the gap, and panel 213 fail by toe kick-in and rotate back.
2. Soil flows through resulting gap between M306 and M307, rotating panel 307.
3. Soil flows through resulting gap between M307 and M308, rotating panel 308, etc.

Sequence of south wall panel movements



Sequence of south and north wall panel movements

Differing restraint imposed by bored piles at north and south walls



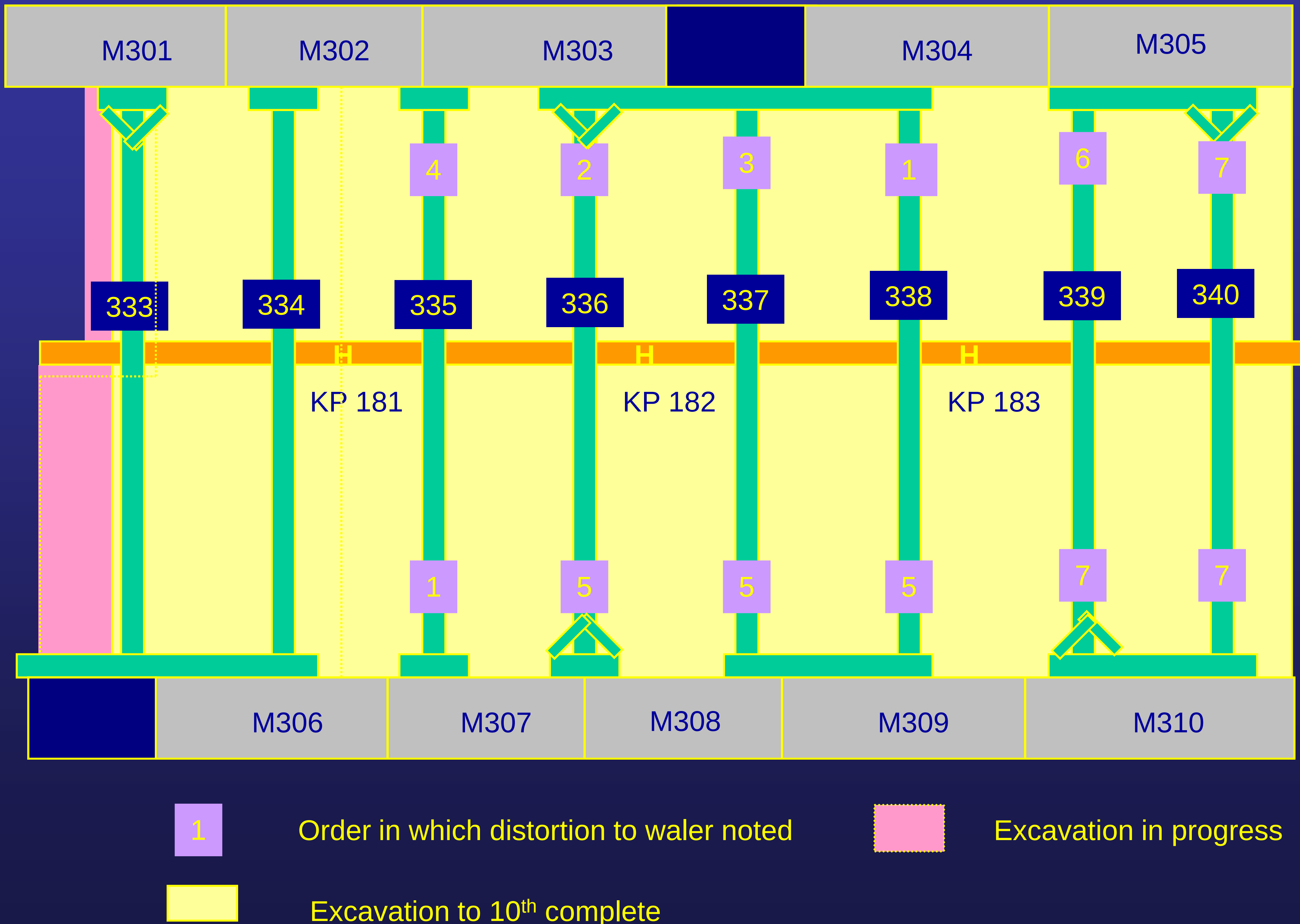
- The bored piles had a major influence on the displacements of the Dwall panels during the collapse and on their post-collapse positions

- The bored piles restricted toe movements on the south side and prevented failure as the buried valley was crossed

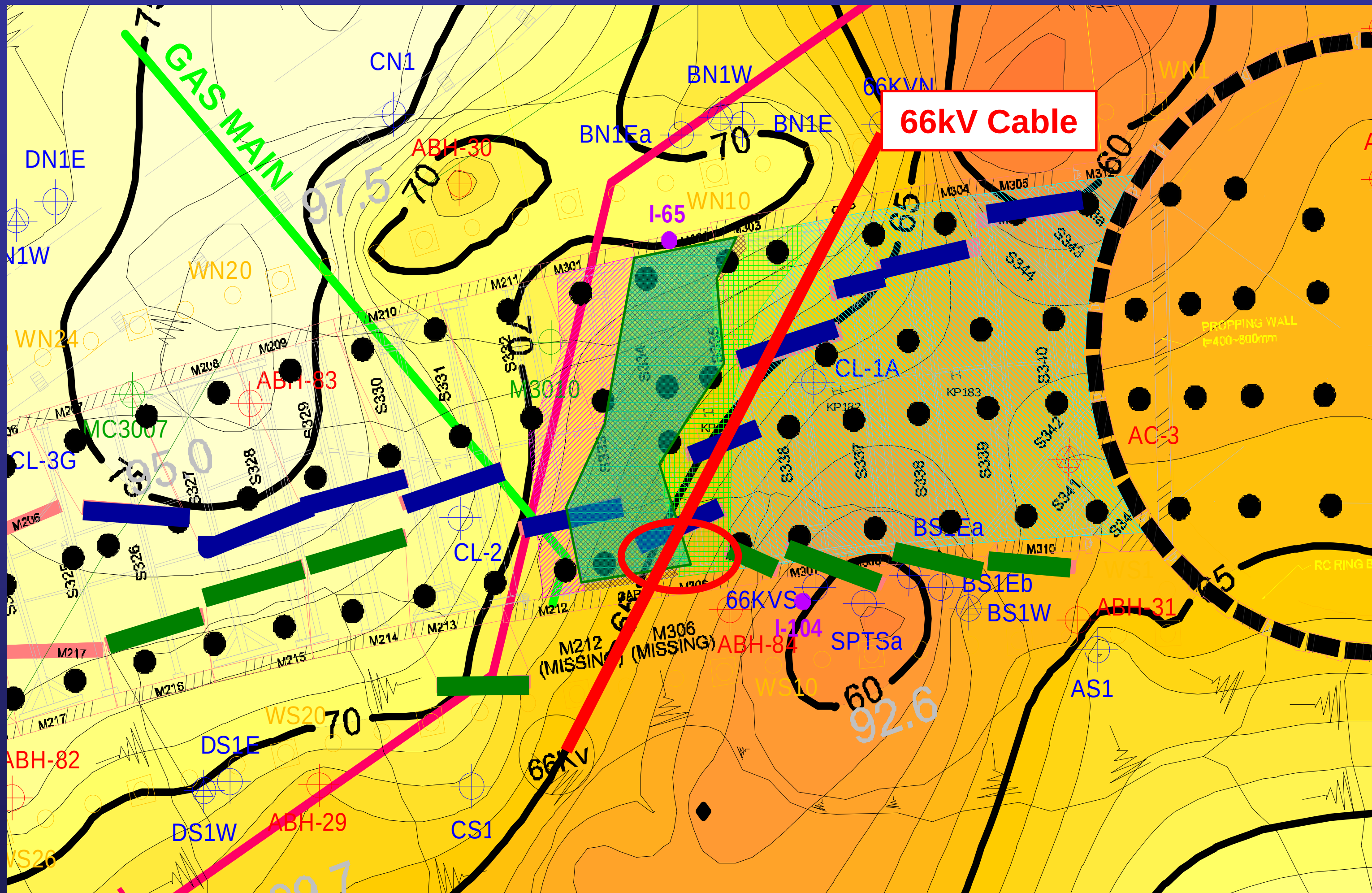
- Loads carried by the bored piles contributed to the under-reading of the strain gauges

Significance of the bored piles

North wall



Location of walers, splays and Dwall gaps

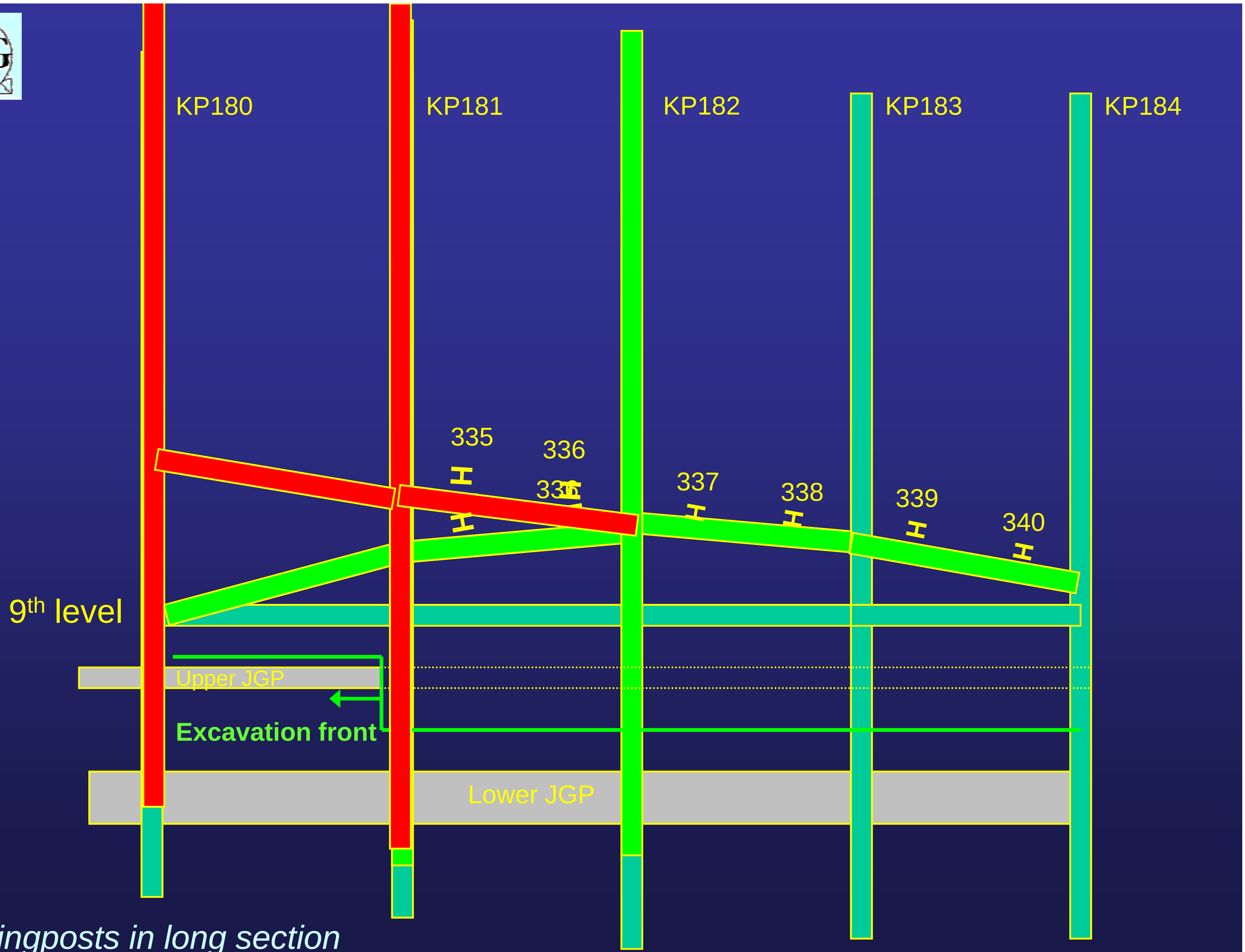


Repositioned bored piles at 66kV crossing

- Upward displacement of KP 180 and 181 accentuated by toe displacement of M306 and M212, where bored piles had been re-positioned and additional toe movement was possible

- Resulting RVD fed back into buried valley

Relative vertical displacement



Kingposts in long section

- C channel stiffened connection undergoes brittle failure
- Critical length of strut and of load in strut result in minimal lateral restraint to connection
- Restraint from rising kingpost results in downward force on connection
- RVD results in reduction in ductility of connection and increase in brittleness
- RVD makes a stable situation with overstress unstable
- RVD results in downward failure of connection

- **RVD was the trigger for the failure**

Failure mode of connection and RVD

Overall conclusions

- The use of Method A in the numerical analyses to model near normally consolidated soils is fundamentally incorrect

- Its use led to under-prediction of wall displacements and bending moments and so to a reduction in the redundancy in the system. The JGP was strained beyond its peak and a plastic hinge formed in the wall as excavation of the sacrificial JGP was underway

Overall conclusions

- There were errors in the design of the strut-waler connection resulting in a design capacity that was 50% of the required capacity where splays were omitted
- The collapse initiated some time after the excavation crossed the buried valley, where forces on the under-designed strut-waler connections would have been a maximum
- An additional perturbation or trigger was necessary to explain the timing of the collapse, the downward failure of the walers at both ends and the trends in the monitoring data

Overall conclusions

- The permanent bored piles in combination with the JGP played a significant role in preventing the collapse as the valley was crossed
- The collapse was triggered when working in the vicinity of the 66kV cable crossing
- At this location, the permanent bored piles had been repositioned and the JGP layout had been modified, allowing the wall toe to kick-in and cause additional uplift of the local kingposts

Overall conclusions

- This additional upward displacement of the kingposts relative to the wall fed back into the system, introducing forced sway failure
- Downward movement of the walls has been predicted by analysis; the potential for relative upward movement of the kingposts has been confirmed by surveys
- Forced sway failure reduced the strain over which the connection remained ductile, increased the brittleness of the connection and allowed a stable situation to become unstable
- Forced sway failure can explain the timing of the collapse, the form of the observed distortions, the trends in the monitoring data, and the speed at which the collapse developed

Overall conclusions

- The collapse was not caused by hydraulic base heave and was not related to poor workmanship
- Wall rotation, which had been linked with inadequate penetration of the wall into the OA, was not the cause of the collapse
- Several factors had to act in combination to cause the collapse

Unforgiving site

- Deepest excavation in marine clay in Singapore –shortcomings in use of Method A not previously apparent because of depth dependence
- Ground conditions – buried valley in OA infilled with soft fluvial and organic clay, rapid variation in depth of marine clay along and across the excavation resulting in an asymmetric section
- Curvature of walls in plan requiring more frequent use of walings
- Presence of 66kVA crossing
- Need to adopt sacrificial JGP layer, removal of which caused step increase in 9th level strut load and step increase in wall settlement

Lessons learnt

- JGP is a brittle material
- The mass properties of JGP need to be more carefully evaluated
- Coring of JGP is not an adequate check
- The use of numerical modelling of soils in design should be carried out by specialists and its incompatibilities with current codes needs to be removed
- The potential for brittle failure of C channel connections must be recognised

Lessons learnt

- Temporary and permanent works should be subject to independent checks
- The effects of relative displacement between kingposts and walls should be considered in the design of strutted excavations
- Forced sway failure and its consequences should be recognised as a potential mechanism in design
- Monitoring did not warn of the impending collapse