

Global Tunneling 31st May 2018

Deep Excavations and Cut and Cover Tunnels

Top – down and Bottom - up

John Endicott
AECOM Fellow

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80 floors Co-development Airport Railway Station below



Hong Kong has many Regulations and “Guidelines”
Building Control, Geotechnical Control

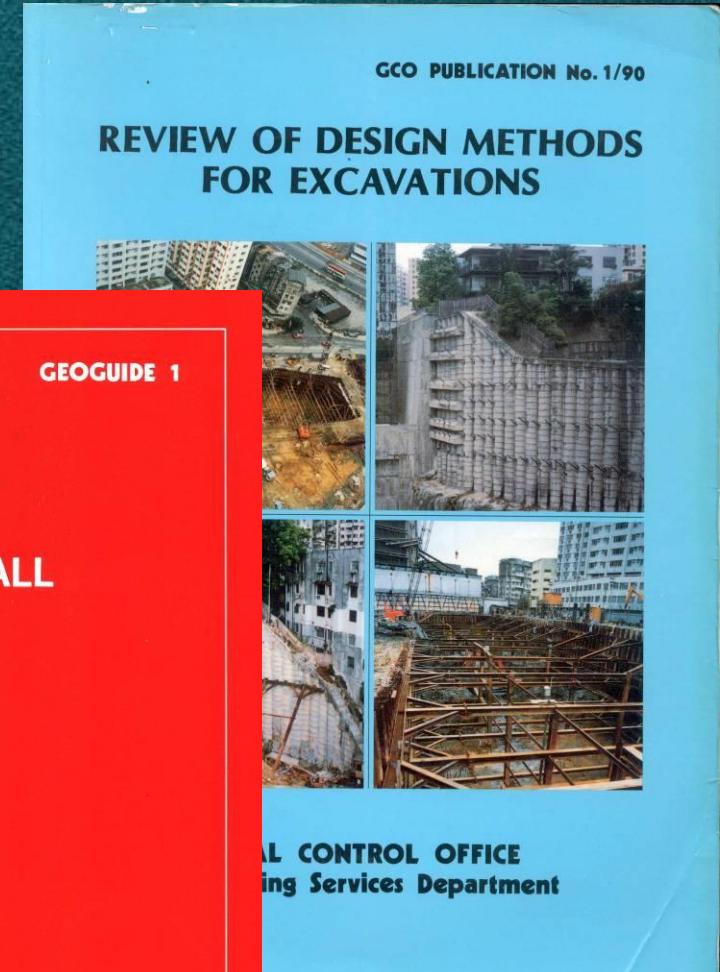
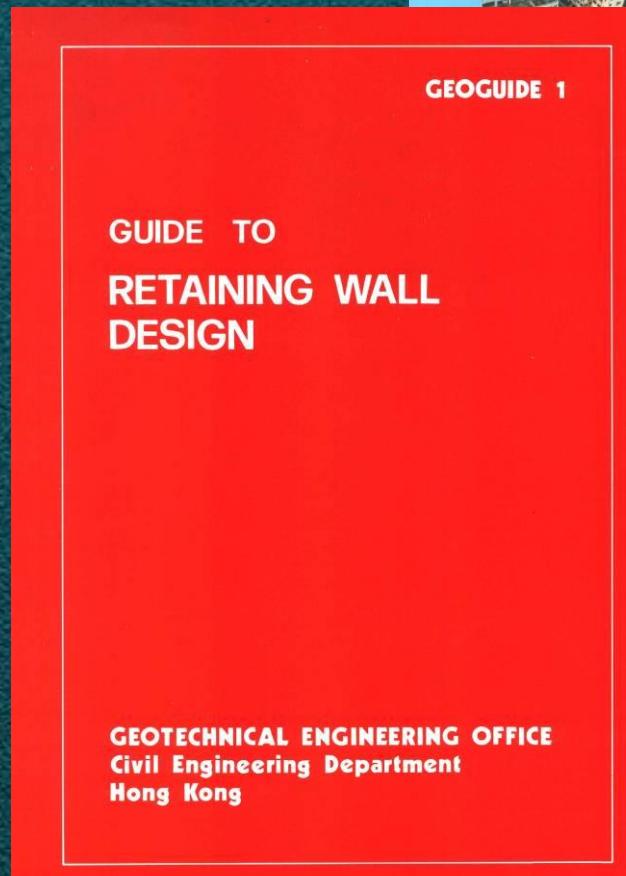
Individual responsibility of Professional Engineers,
Registered Geotechnical Engineers
Imprisonment: custodial sentence

Not yet Registered Tunnel Engineers

Singapore has similar Building Control

Design Codes and Guides

- CIRIA 104
- BS 8002
- DD ENV
- BD 42/00
- Piling handbook
- CIRIA R185
- CIRIA C517
- CIRIA SP95
- GEOGUIDE 1 2nd Ed.
- GCO Publication No. 1/90
- C 580 Limit State



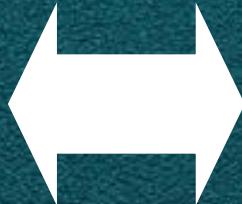
Web-Based Real Time Reporting



dataloggers
with solar panel



Instruments
(eg.strain
gauges on
struts)



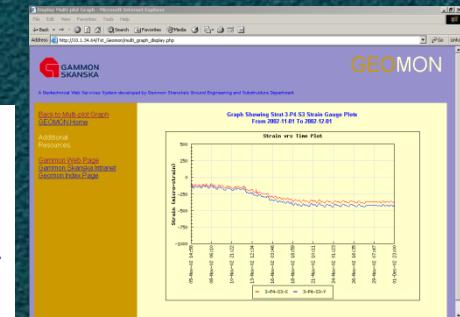
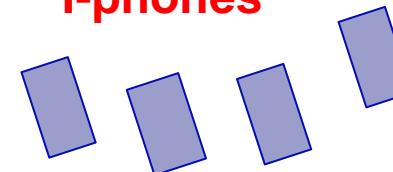
Server

Internet



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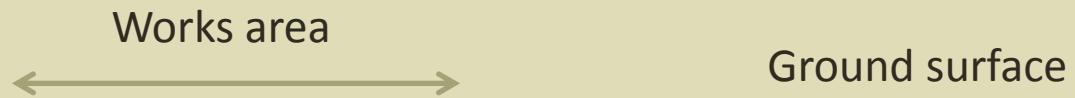
i-phones



Real time
monitoring
and alerts

Top-Down Method of deep excavation and construction

Stage 0



Brief summary
of sequence

Top-Down Method of deep excavation and construction

Stage 1

Install diaphragm walls



Cutter for
excavating

Diaphragm
Walls

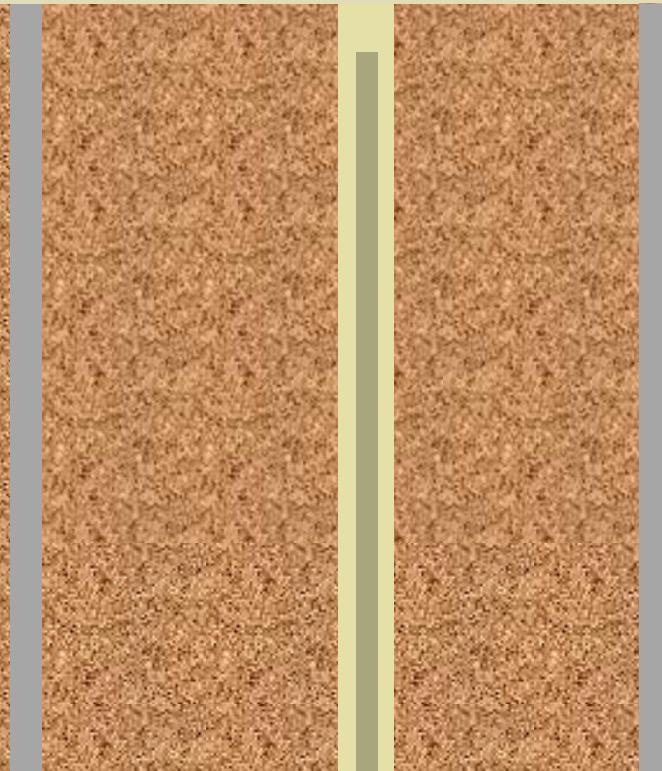


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Top-Down Method of deep excavation and construction

Stage 1

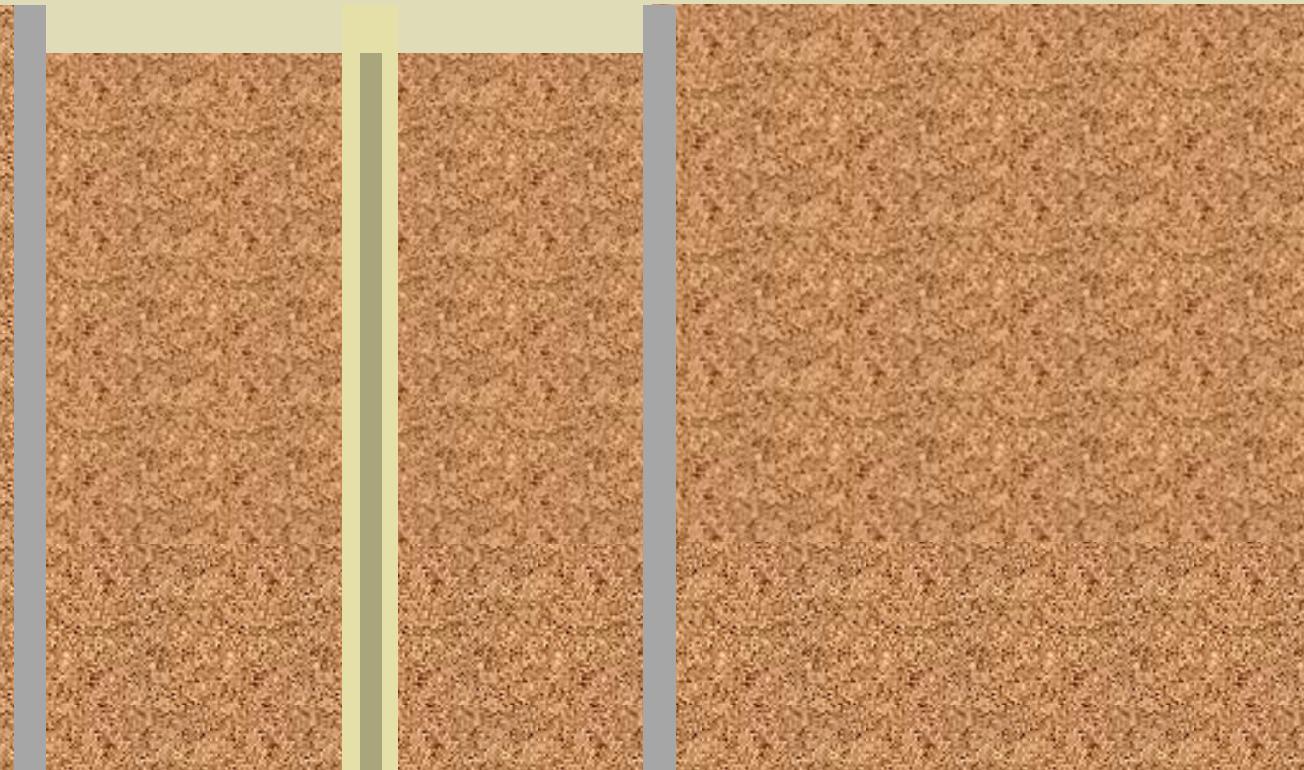
Install diaphragm walls and plunge columns



Top-Down Method of deep excavation and construction

Stage 2

Excavate for top deck/slab



Sometimes
the
top slab

is
**below
ground**

and is
**later
backfilled**



Top-Down Method of deep excavation and construction

Stage 2

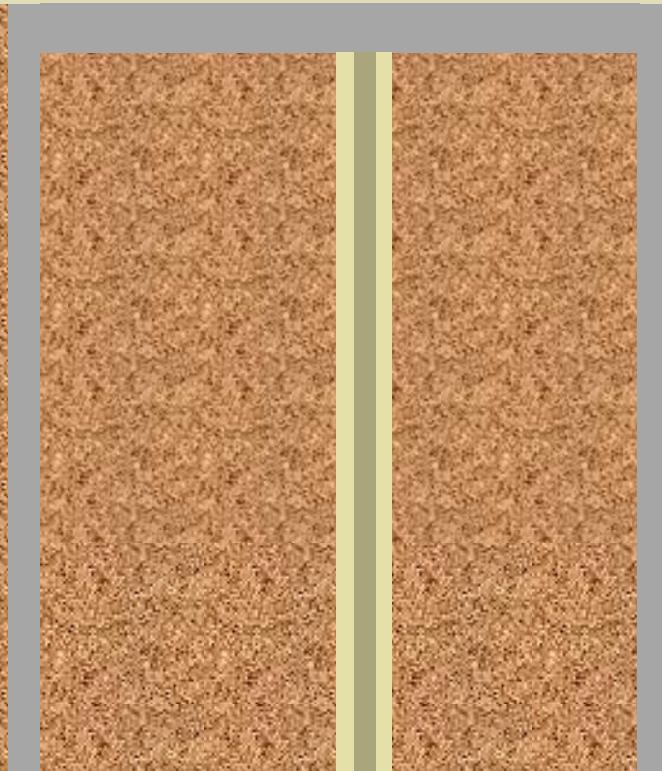
Excavate for top deck/slab



Top-Down Method of deep excavation and construction

Stage 4

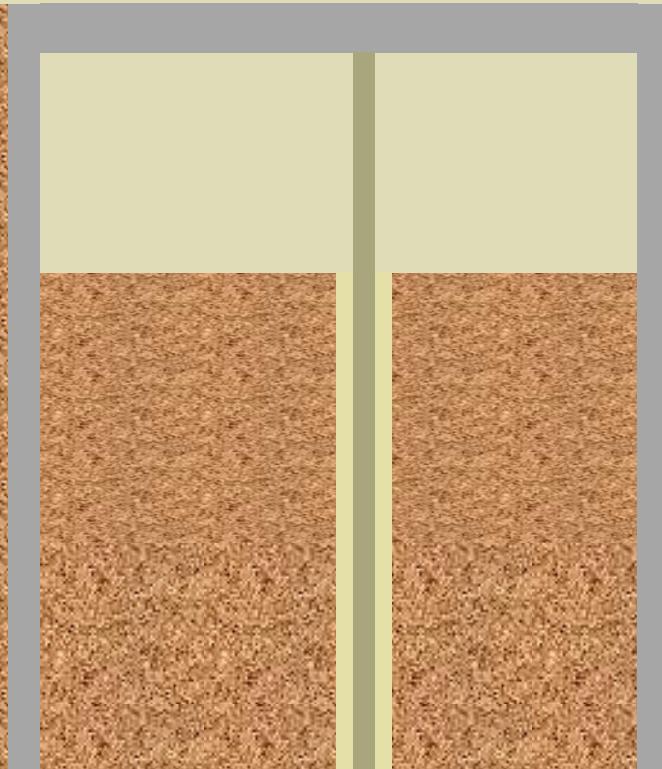
Construct top deck/slab



Top-Down Method of deep excavation and construction

Stage 6

Excavate to First level



Top down creates a lot of protected space on site that is often under-utilised

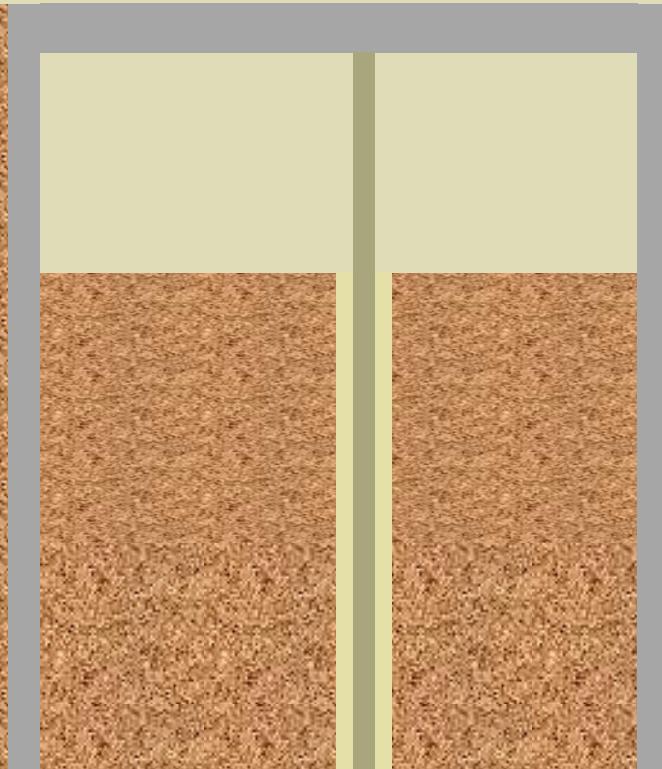


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Top-Down Method of deep excavation and construction

Stage 6

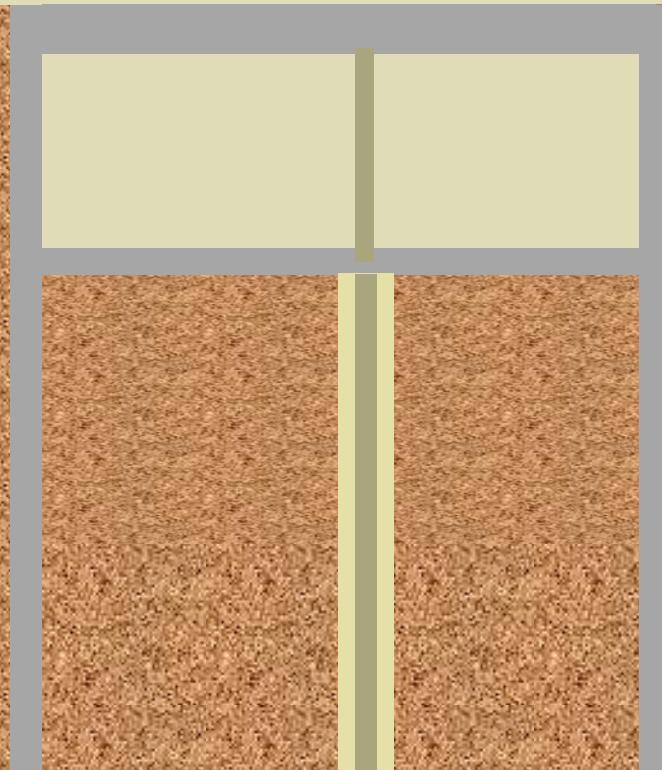
Excavate to First level



Top-Down Method of deep excavation and construction

Stage 6

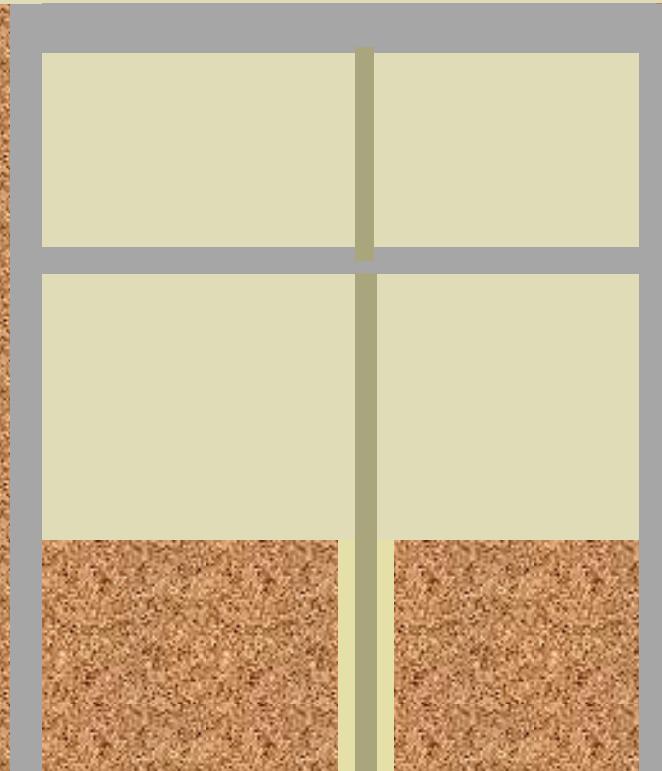
Construct first level deck/slab



Top-Down Method of deep excavation and construction

Stage 6

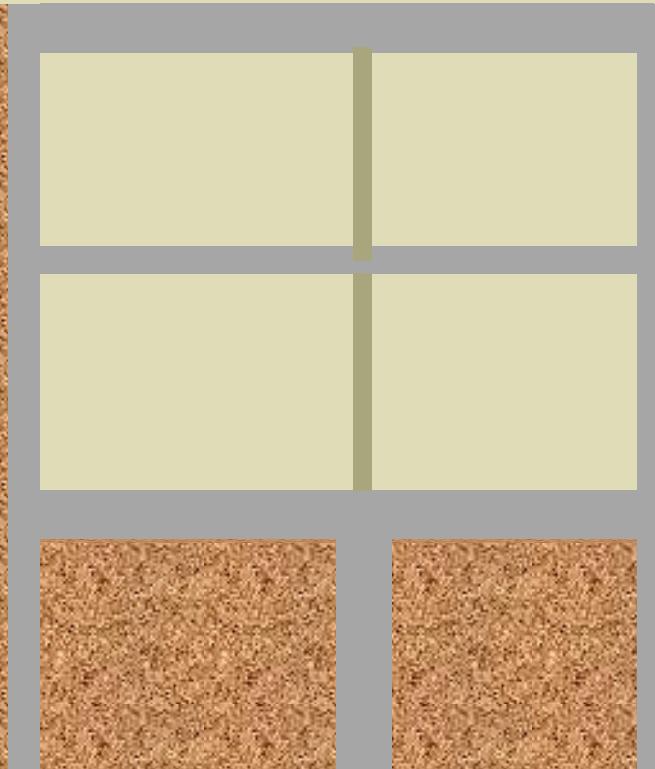
Construct first level deck/slab



Top-Down Method of deep excavation and construction

Stage 8

Construct bottom slab



Top-down: Permanent Diaphragm wall and Skin wall



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PLAXIS and others

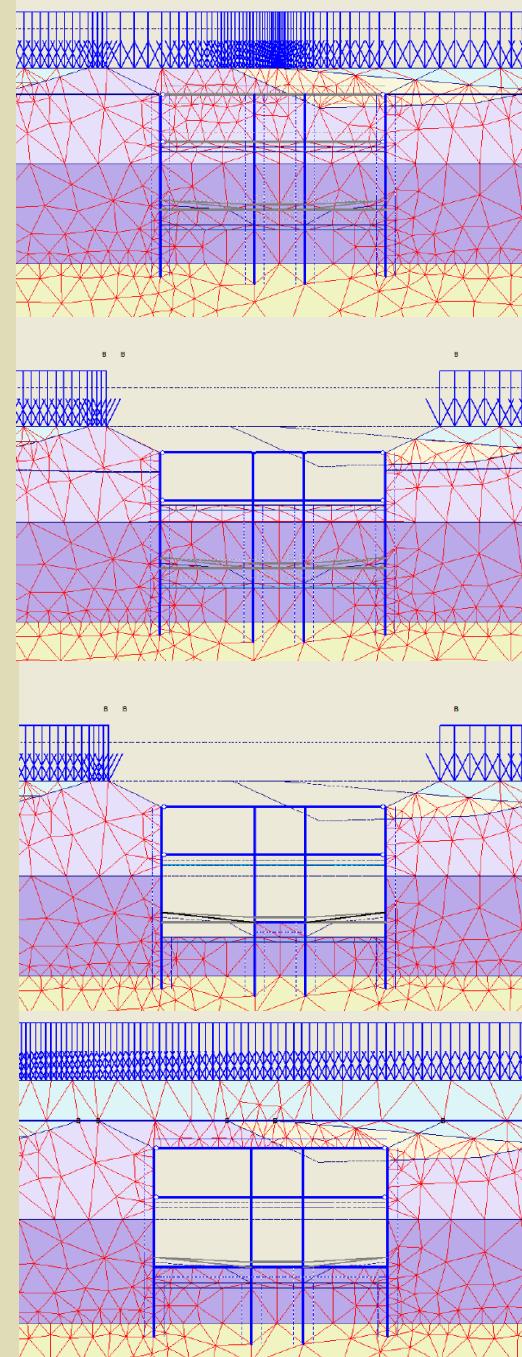
can model the sequence in many stages for you

Excavate to top slab level and install top slab

Excavate to first level and construct first slab

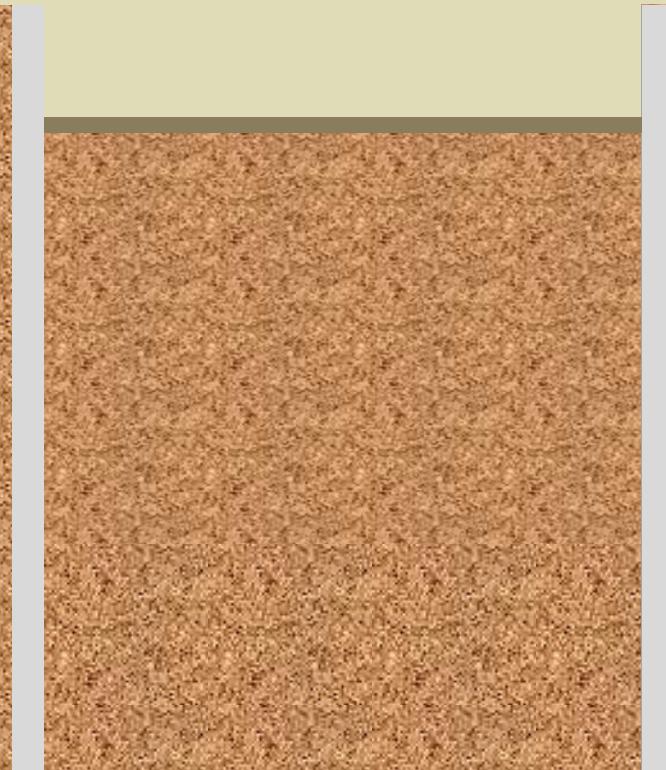
Excavate to final level and construct bottom slab

Backfill on roof



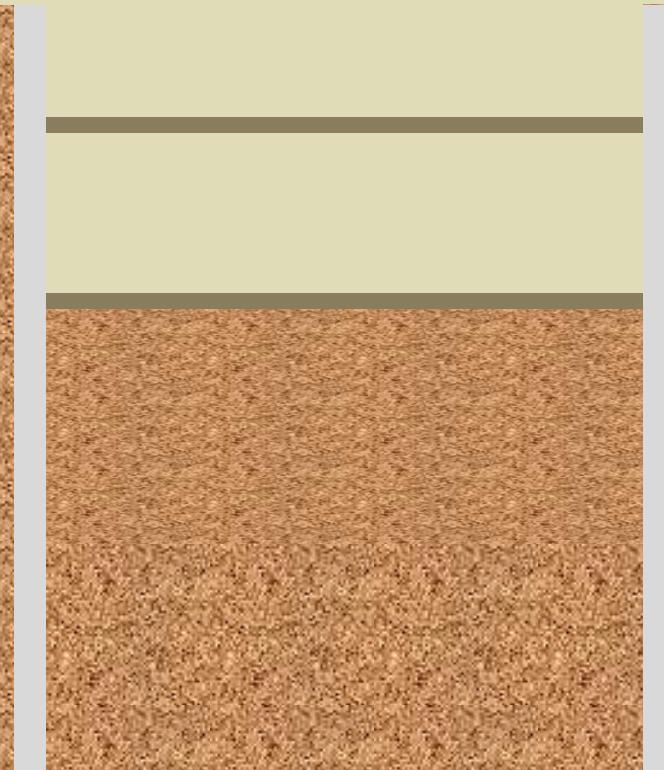
Bottom-up Method of deep excavation and construction Stage 3

Install temporary walls, excavate for, and install top temporary strut.



Bottom-up Method of deep excavation and construction Stage 5

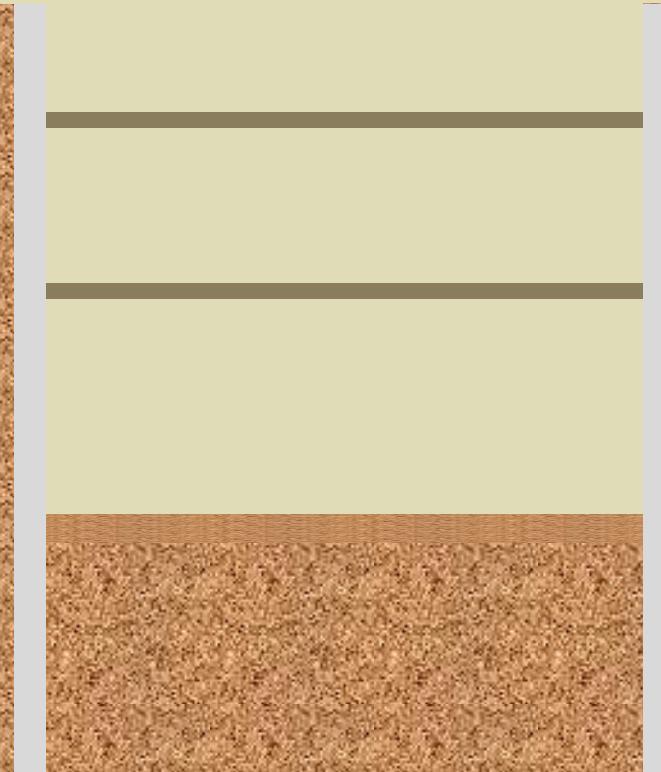
Excavate to second level and install second level temporary strut



Bottom-up Method of deep excavation and construction

Stage 8

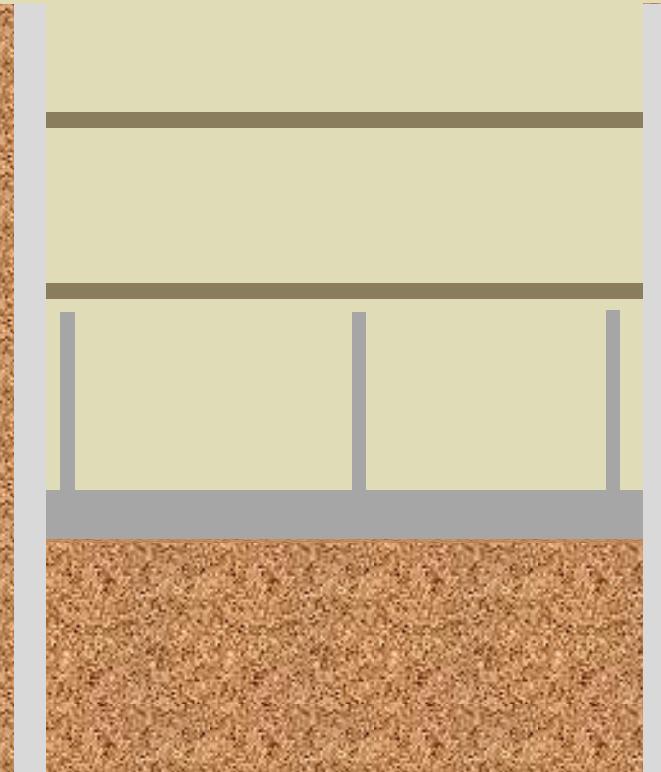
Excavate to final level



Bottom-up Method of deep excavation and construction

Stage 9

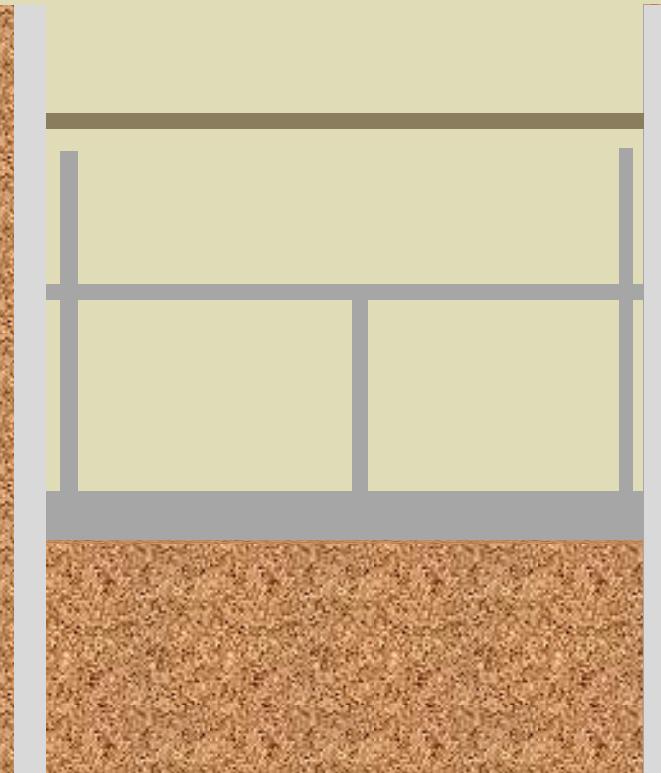
Cast base and first lift of permanent walls and columns



Bottom-up Method of deep excavation and construction

Stage 11

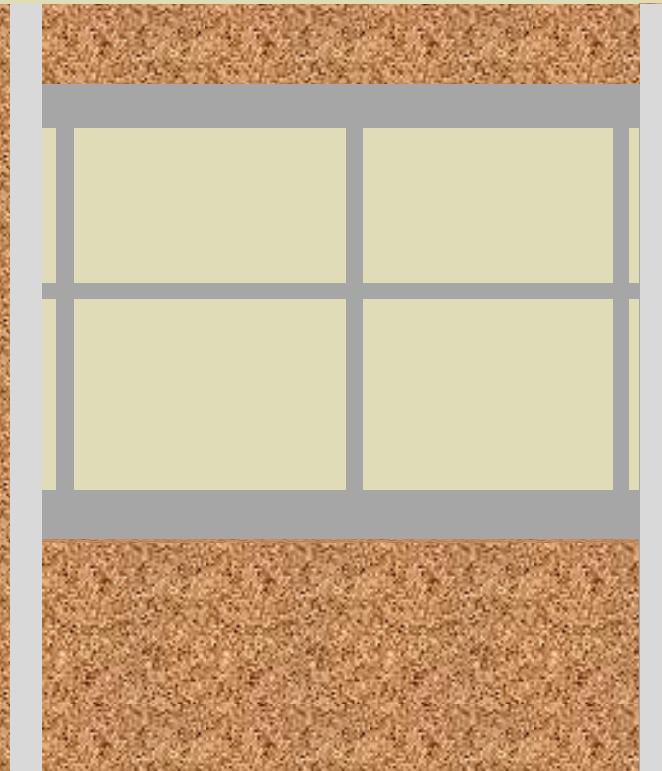
Remove second strut, cast concourse floor and second lift of permanent walls.



Bottom-up Method of deep excavation and construction

Stage 13

Remove top strut, cast walls, columns and roof and backfill.



Examples

A variety of applications

The methods are not new

Bottom-up construction

Temporary steel sheet pile walls and

Struts

or

Tie-back anchors



1976

Bottom-up construction

Permanent diaphragm walls, temporary strutting



Re-usable
hired struts
from
Yong Nam

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Top-down:

Permanent primary roof beams at every plunged column position

Hand excavated caisson walls

1976



Primary roof beams

used as casting beds for pre-tensioned Tee-beams to be placed between primary beams.



Top-down or bottom-up, Benefits?

Top-down

Early completion and access at ground level

No temporary shoring (or only a little)

No formwork for floors and roof

Cheaper to construct

Contractor's alternative

Bottom-up

Easier to design

Better waterproofing

More access for cranes

Reference design

Top-down or bottom-up, Dis-Benefits?

Top-down

Less access below slabs during excavation

Plunge columns

More complicated to design

Contractor's alternative

Bottom-up

More expensive to construct

Usually slower

Reference design

Case 1

Top-down Basement

5 level basement provided a ground level slab with large site works area at ground level and

Bottom-up Superstructure

Simultaneous construction



Wynn's casino, Macau.

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Large ground level platform with several openings for access to basement



Mucking out basement



5th basement

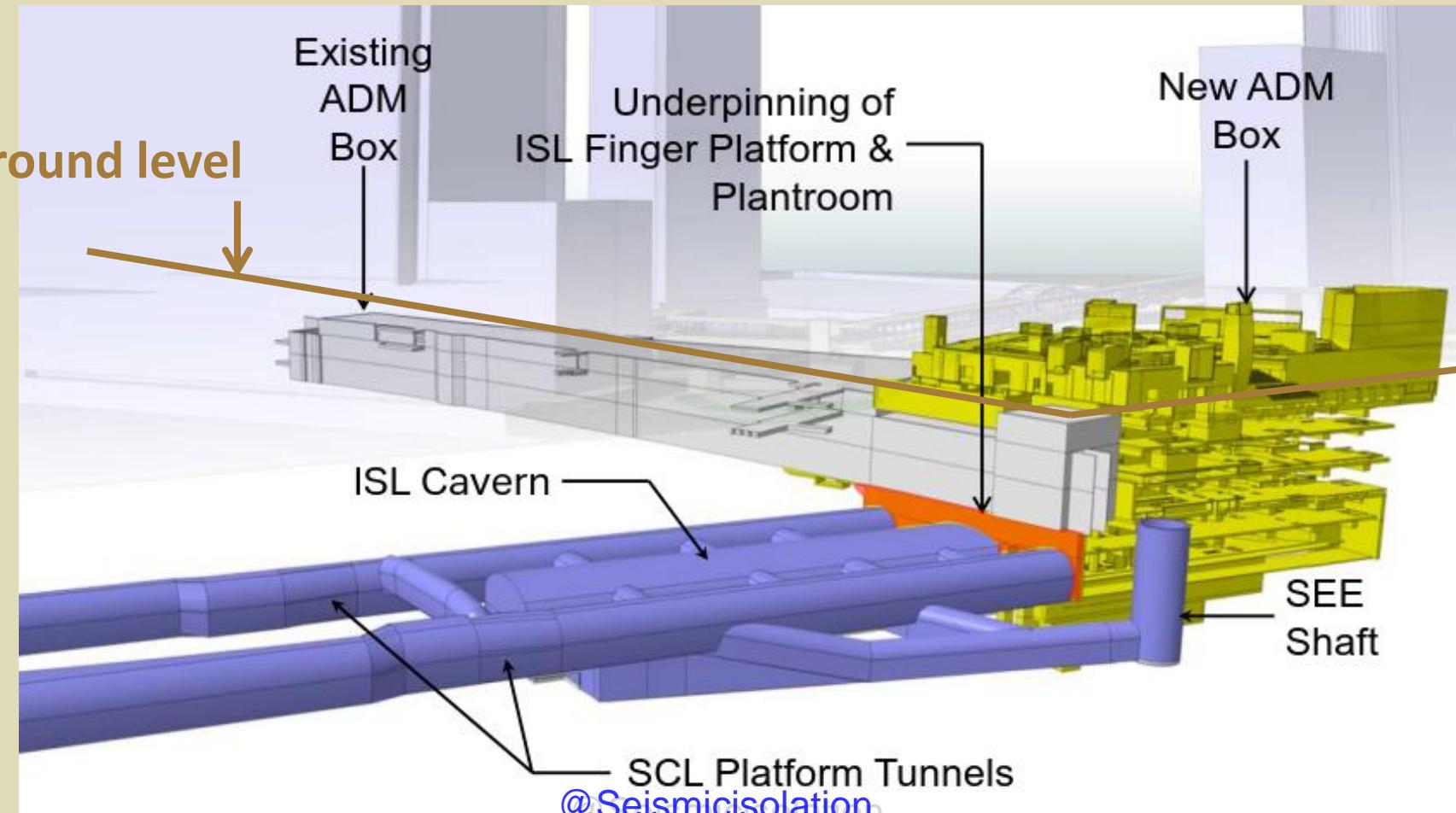
Case 2 SIL at Admiralty Station - 3 Lines

Undermining two lines and end of junction station



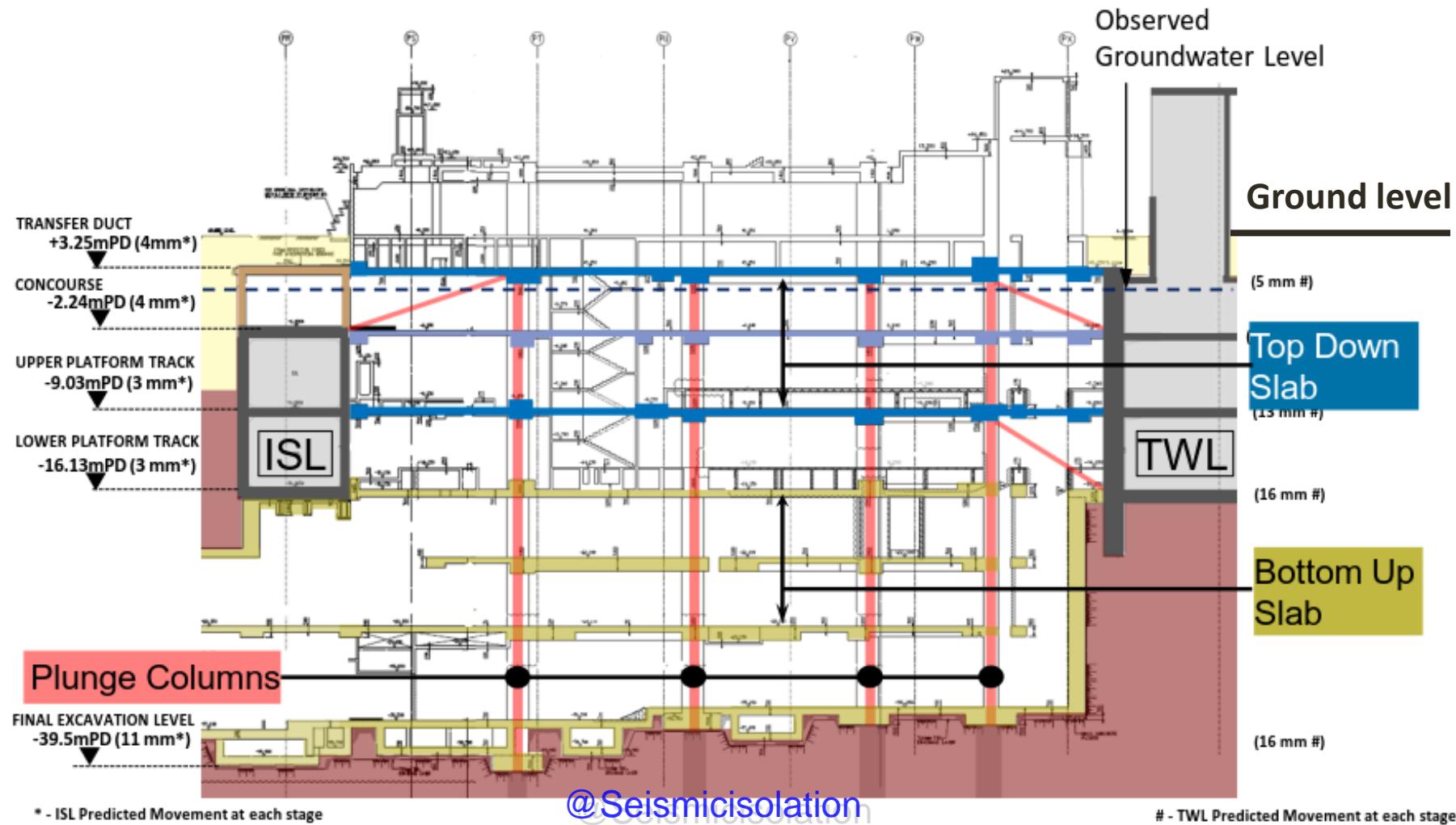
Complexity

Schematic of physical interfaces

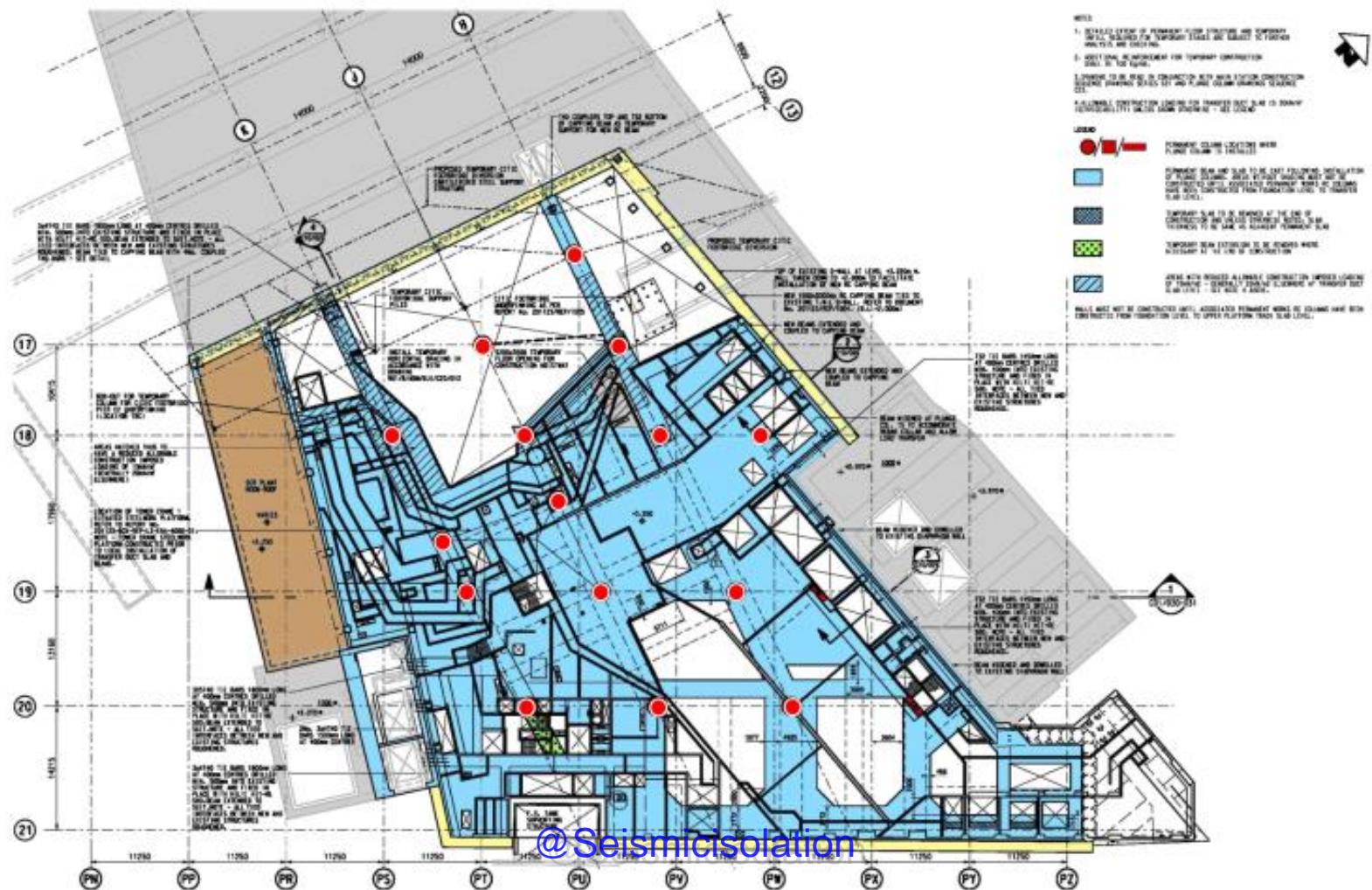


Hybrid:

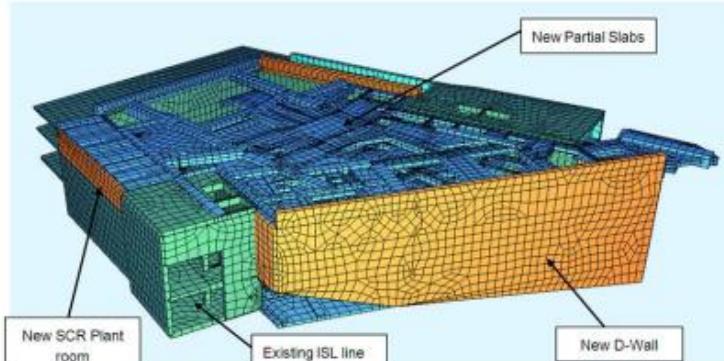
3 floors top-down then 4 floors bottom-up



Top-down involves incomplete slabs for access



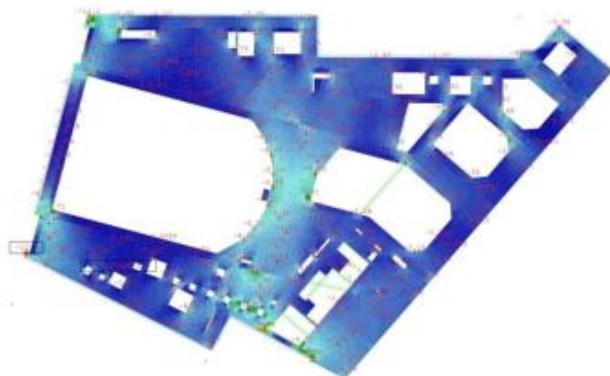
3-D modelling of structure at several stages



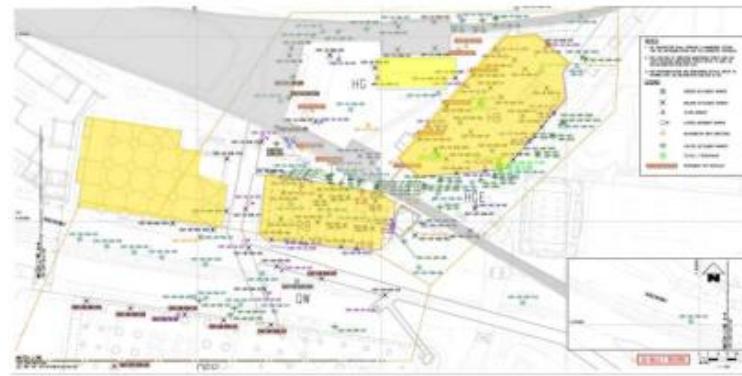
Soil Structure Interaction FE Models



Ground Settlement Predictions



Operational Infrastructure Verification



AAA / Instrumentation & Monitoring

Top-down first

then

Bottom-up



Permanent structural elements used
for temporary lateral bracing



Plunged columns and blinding for Bottom-up
construction

The excavation is quite big

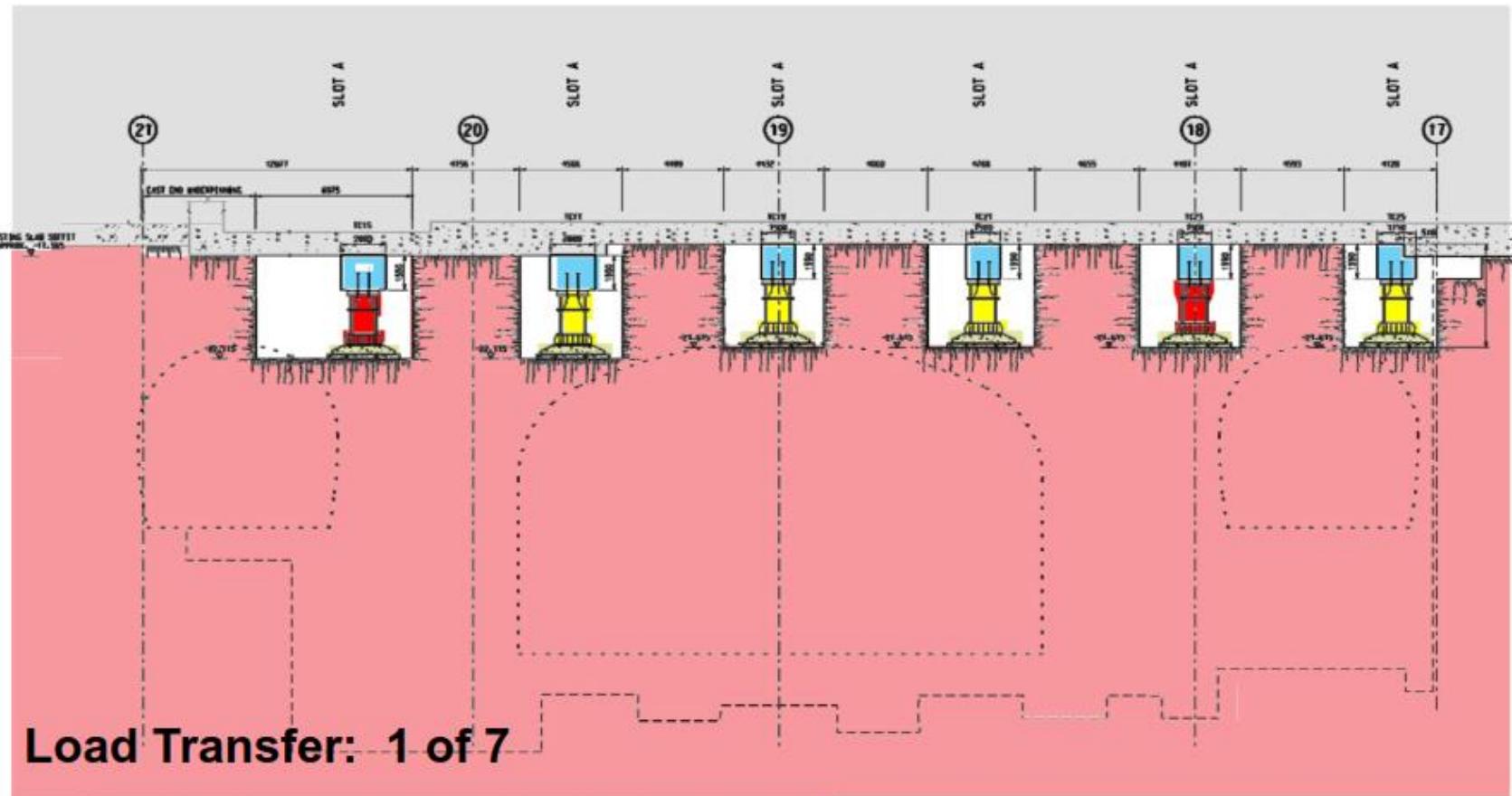


Underpinning for excavation beneath live tracks.

Admiralty Station Expansion

Step 1 of 12

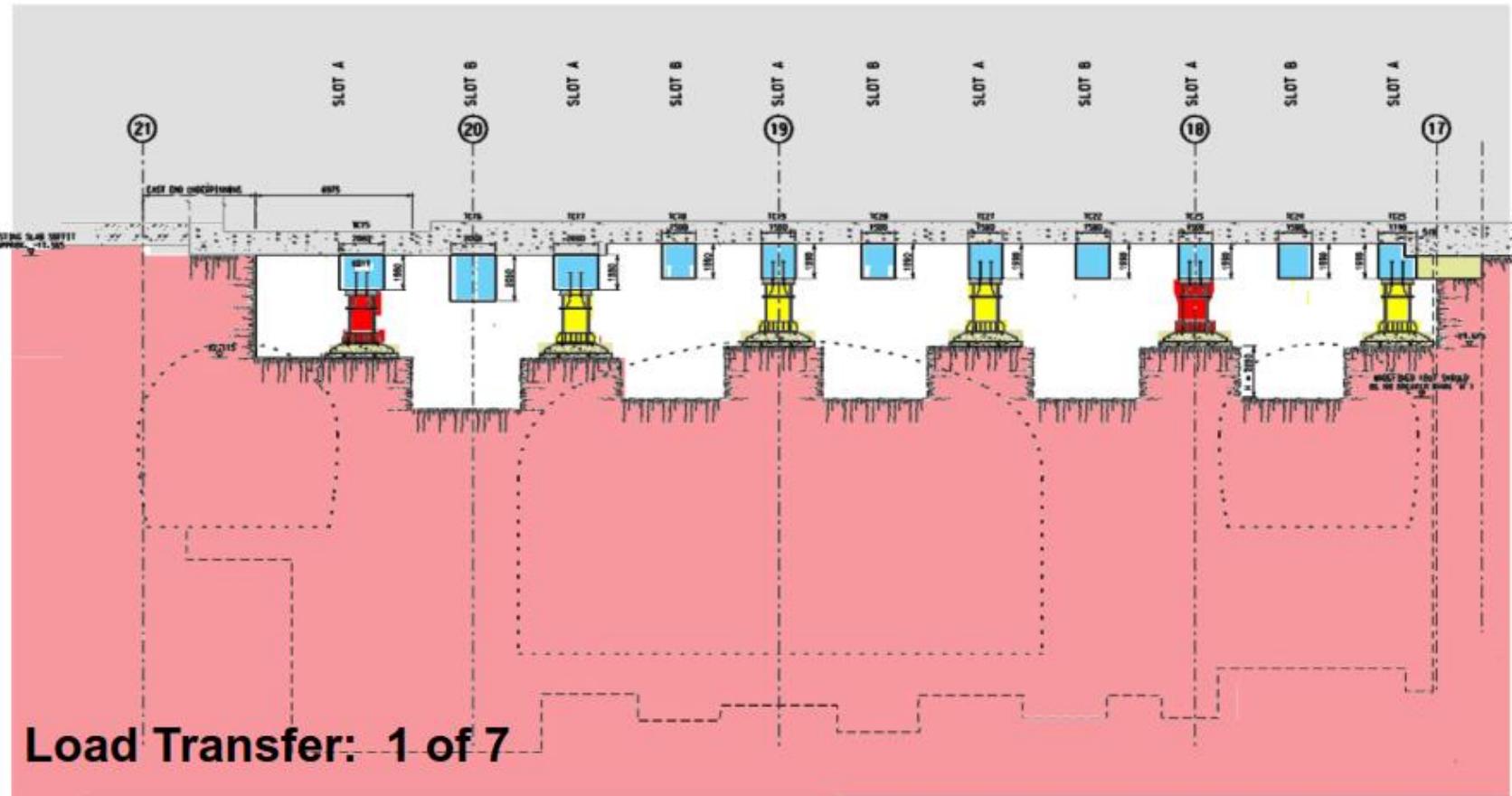
Underpinning for excavation beneath live tracks.



Underpinning ISL Finger Platform Tunnel

Sequence of Works

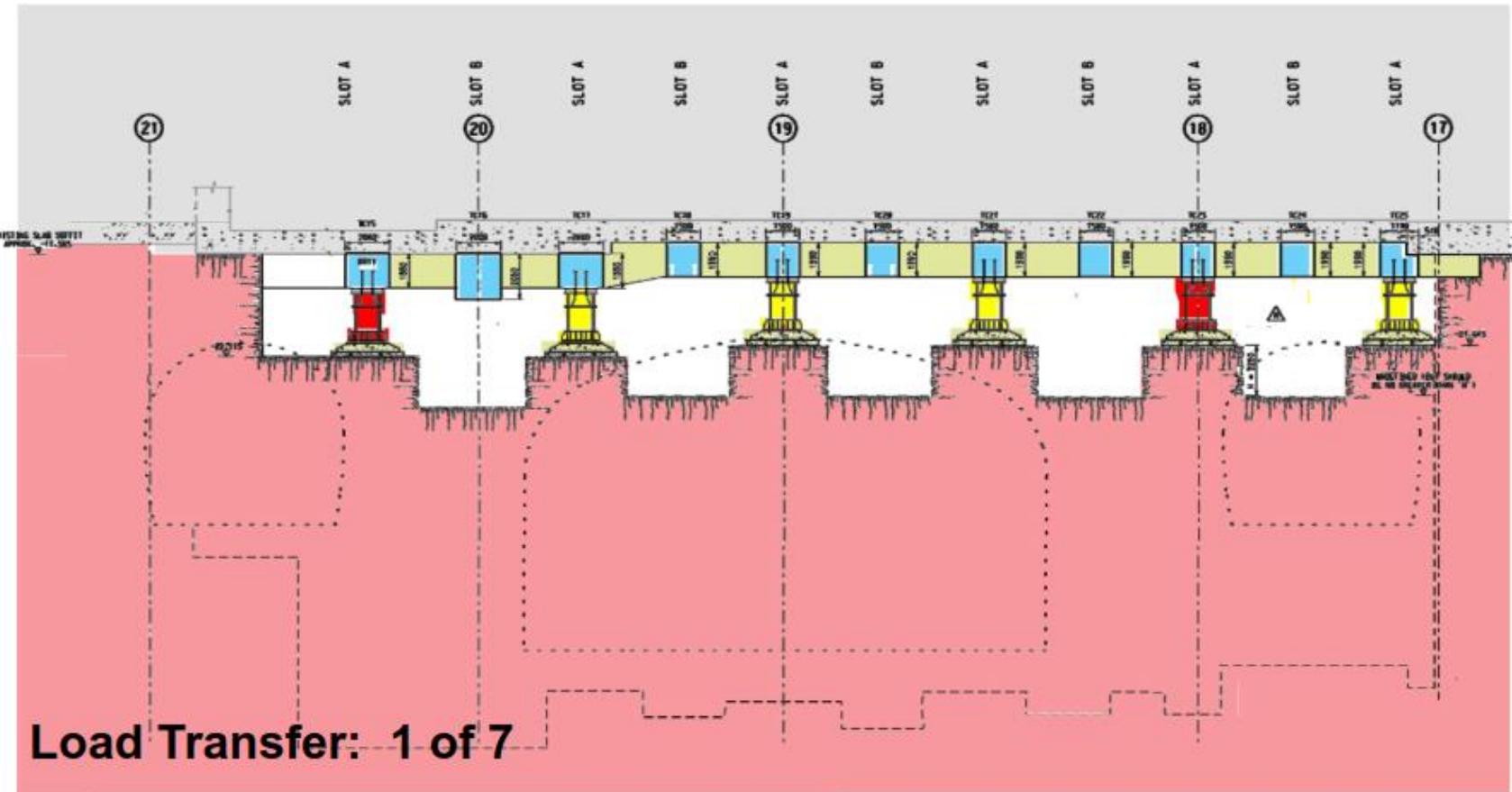
Step 2 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

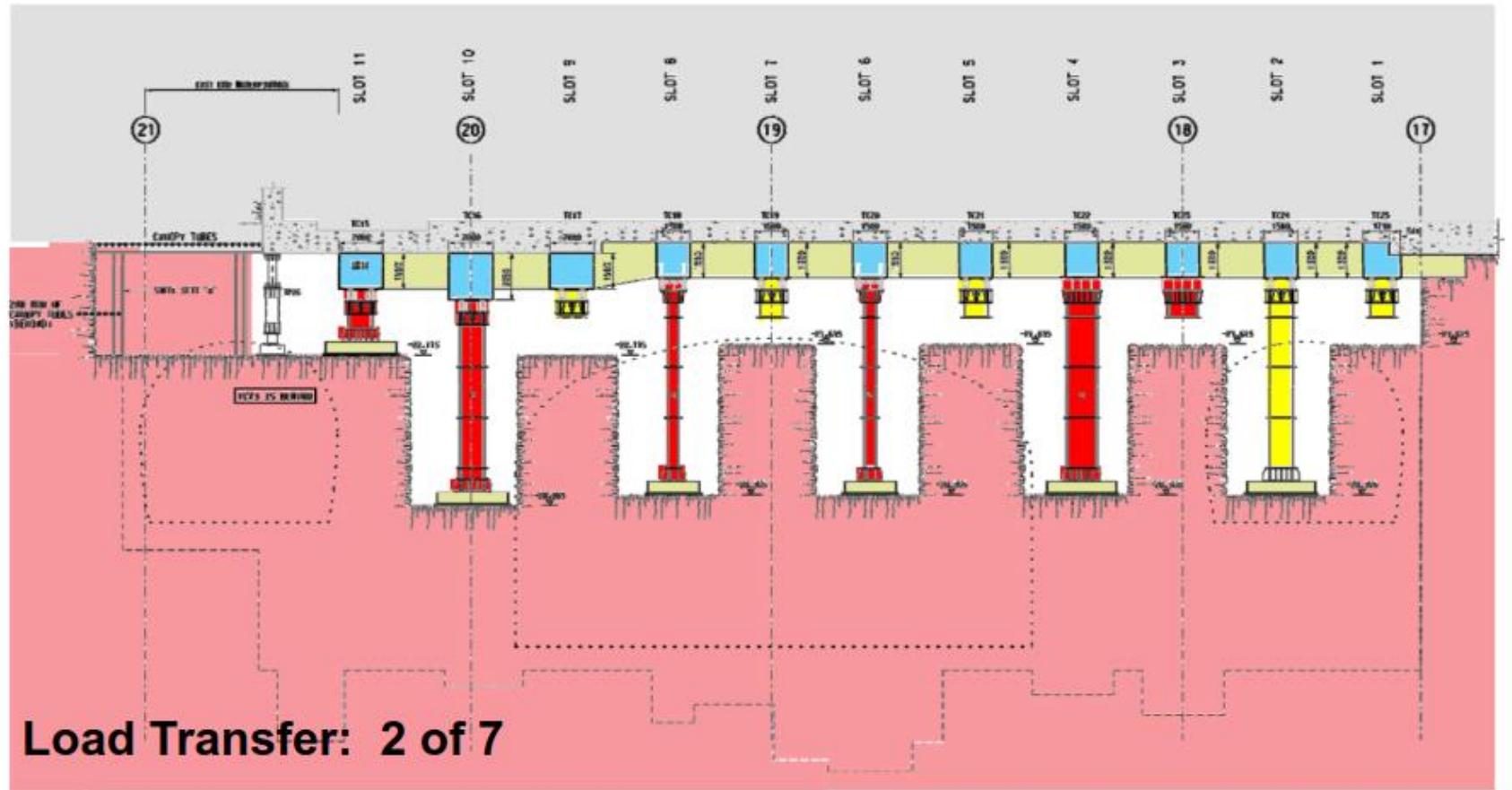
Step 3 of 12



Load Transfer: 1 of 7

Underpinning ISL Finger Platform Tunnel Sequence of Works

Step 4 of 12



Admiralty Station Expansion

May, 2016

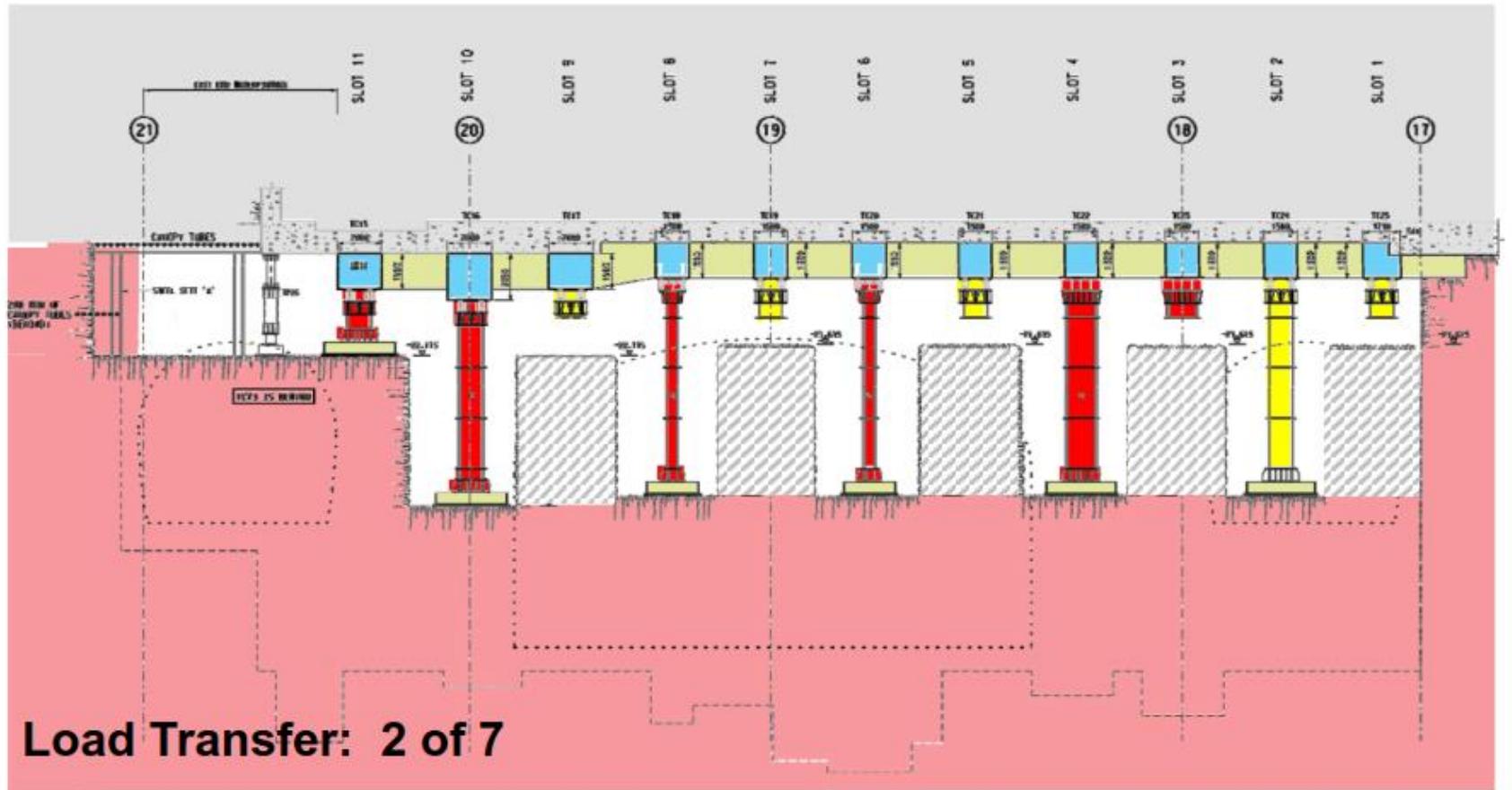
@Seismicisolation Page 24

benaim

Underpinning ISL Finger Platform Tunnel

Sequence of Works

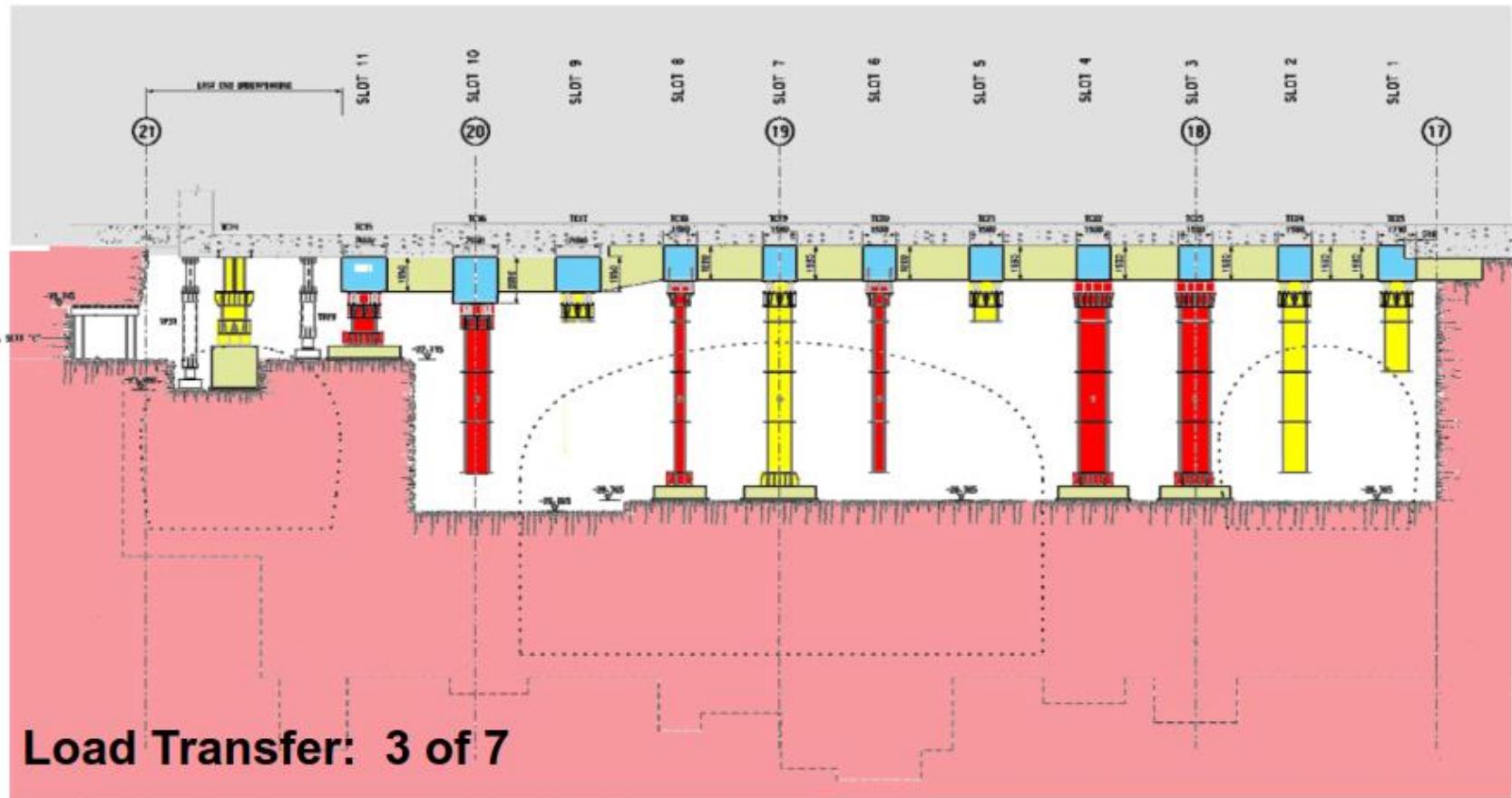
Step 5 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

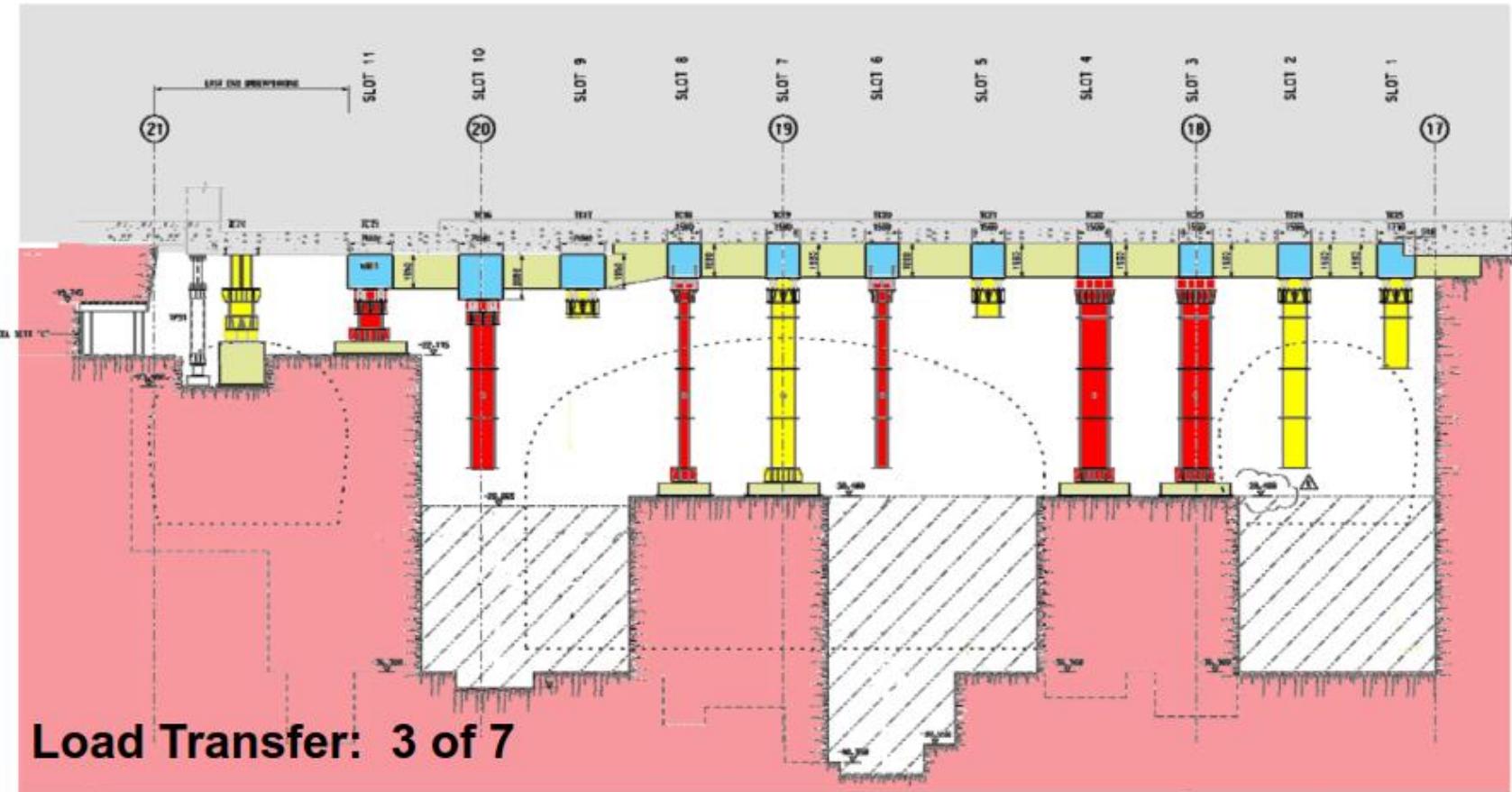
Step 6 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

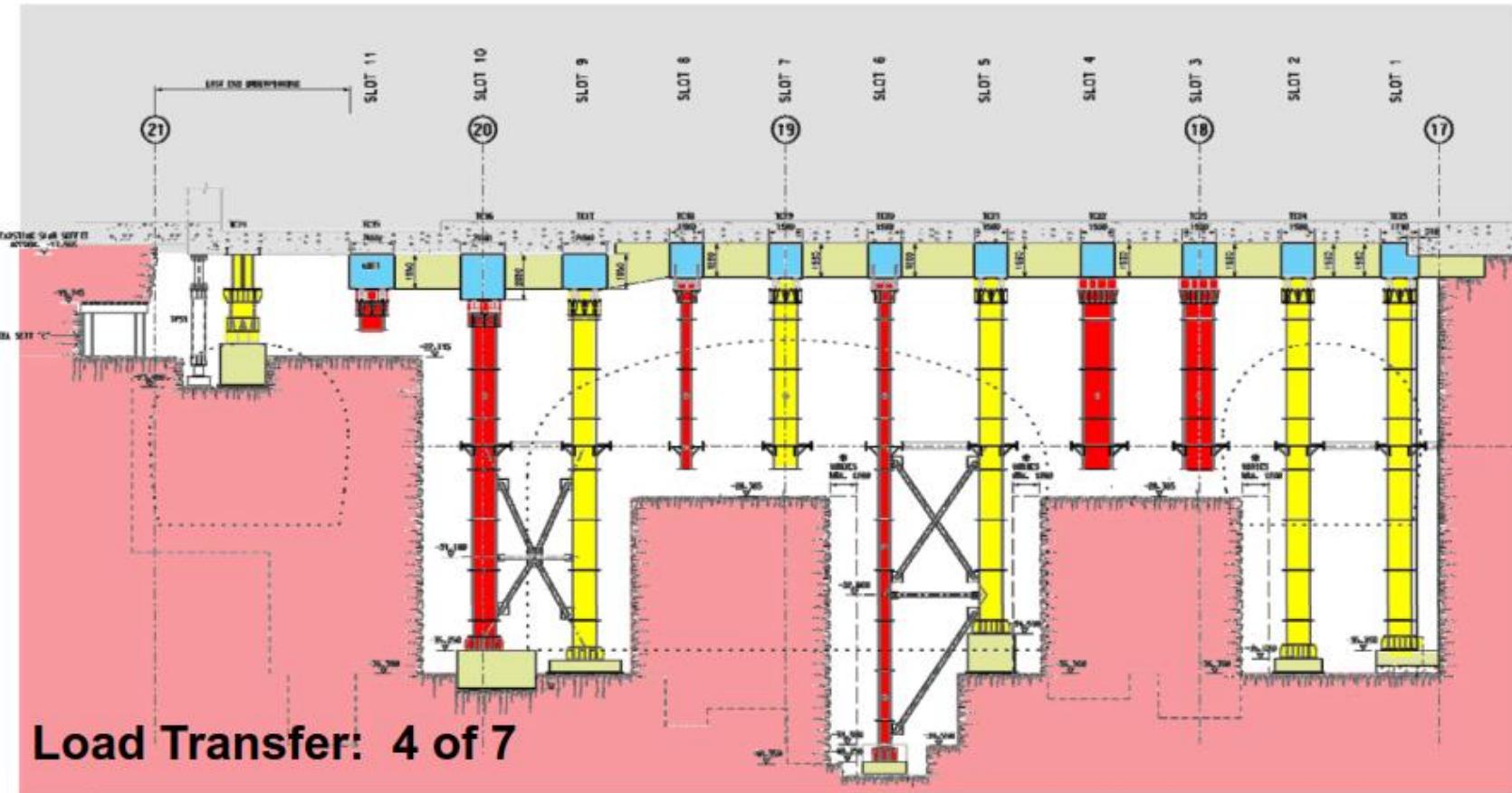
Step 7 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

Step 8 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

Step 9 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

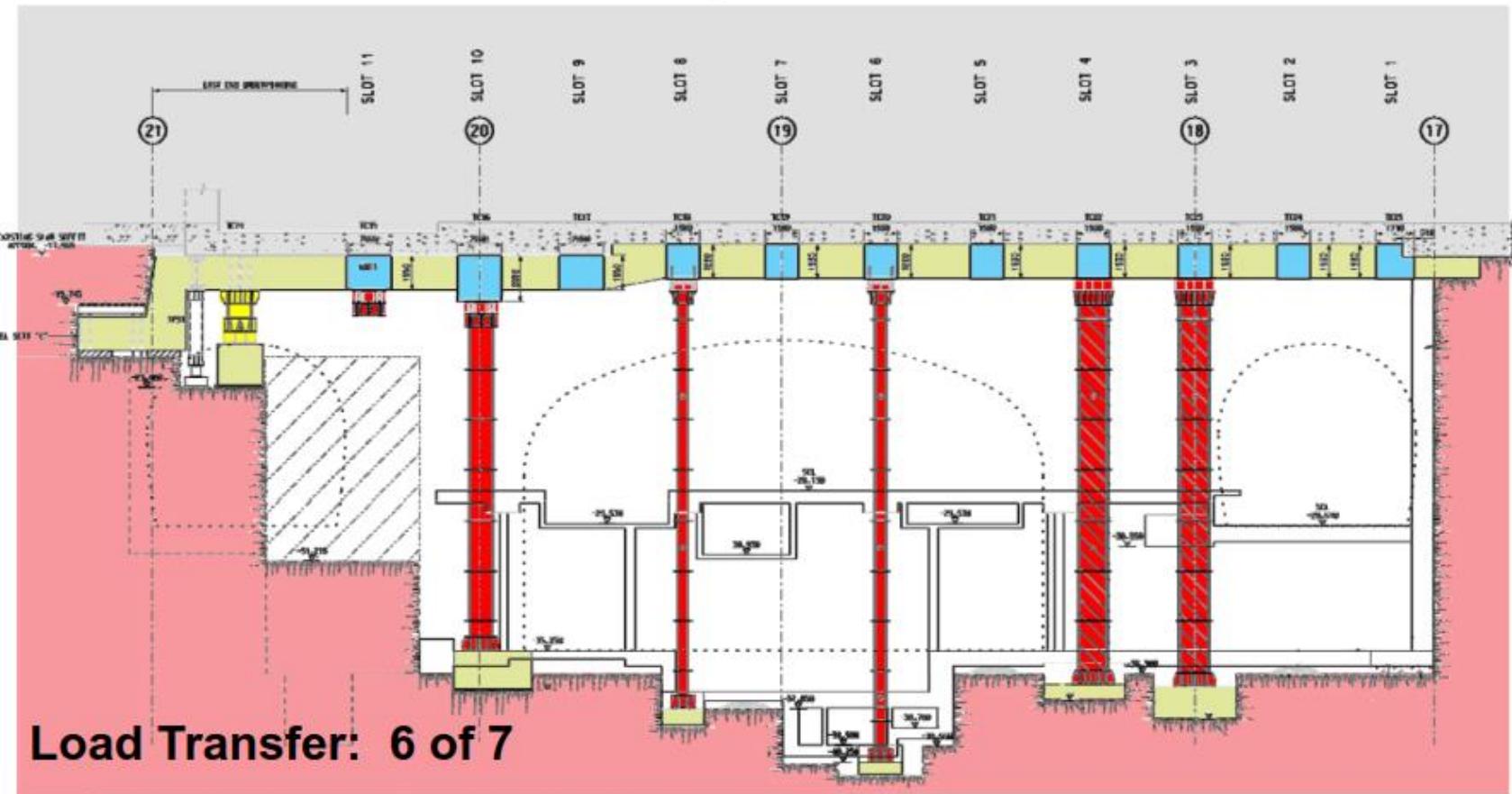
Step 10 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

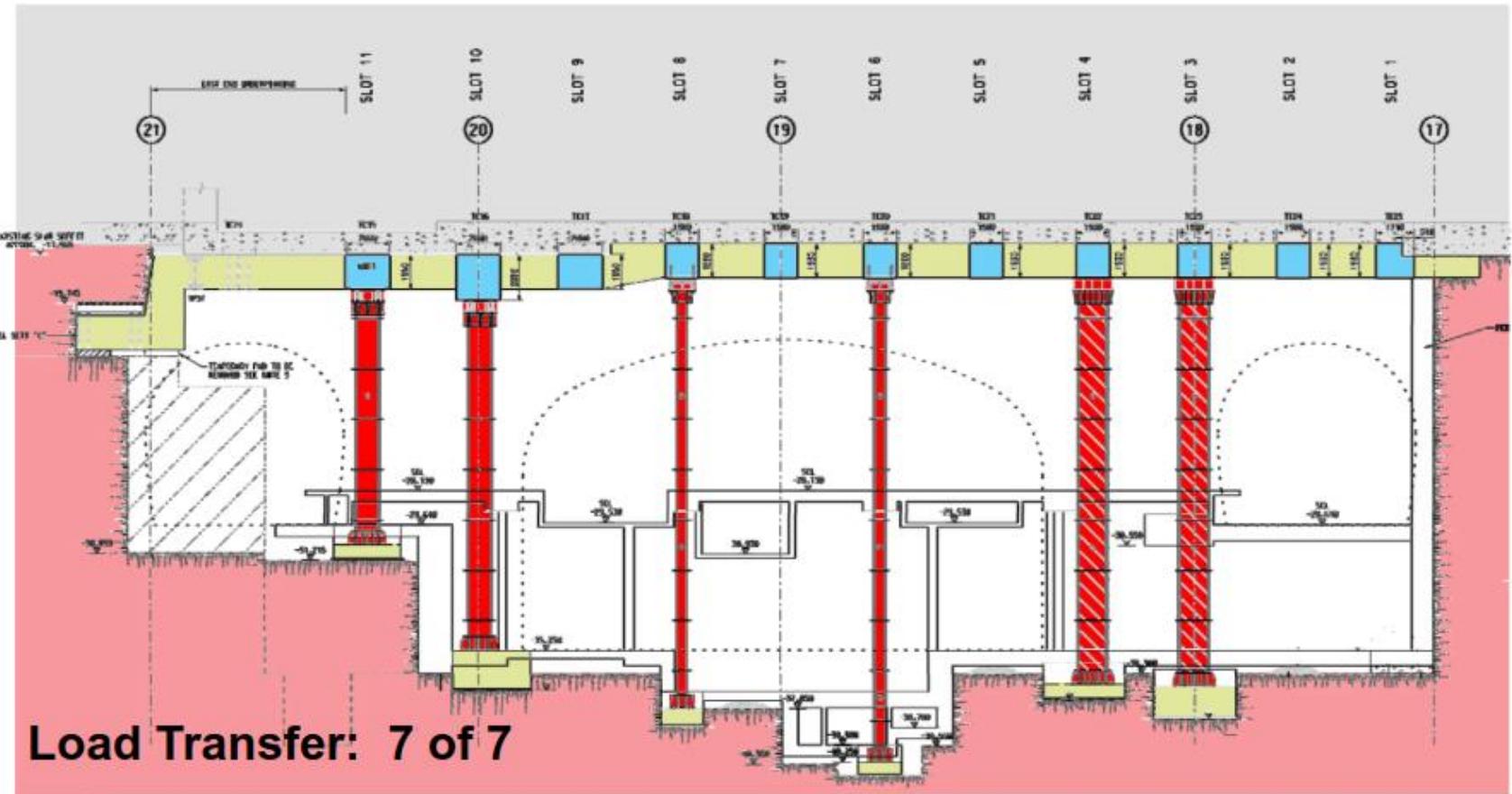
Step 11 of 12



Underpinning ISL Finger Platform Tunnel

Sequence of Works

Step 12 of 12



Underpinning ISL Finger Platform Tunnel

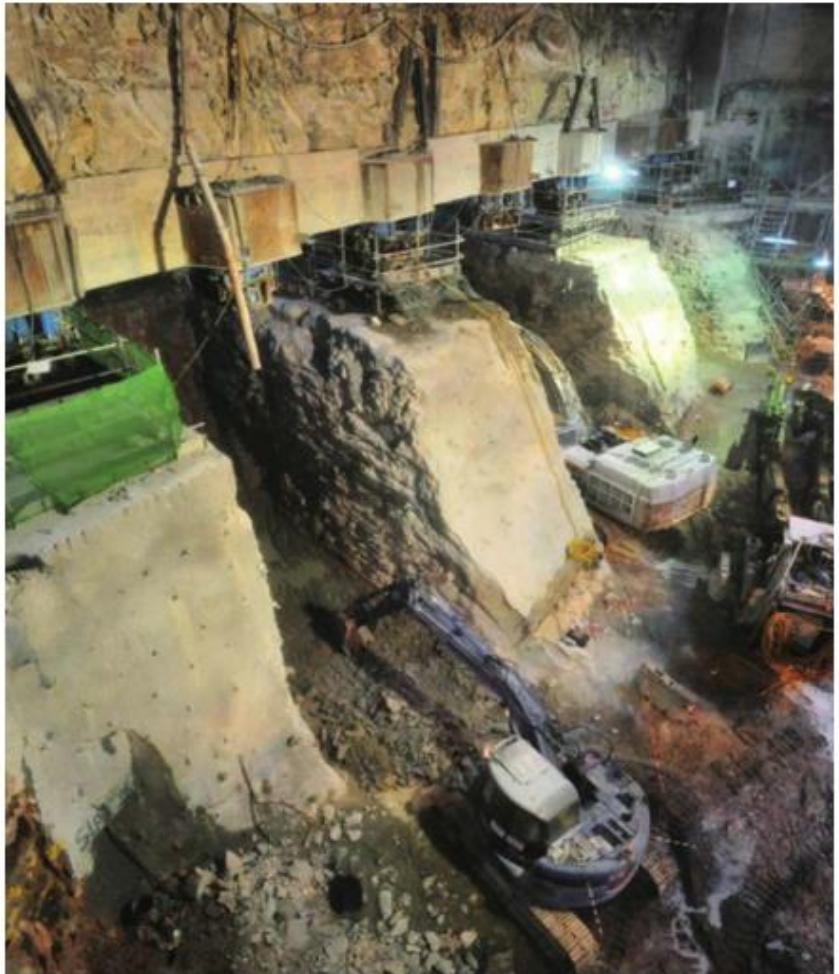
Jack / Load Cell Station



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Underpinning ISL Finger Platform Tunnel

Excavating Slots



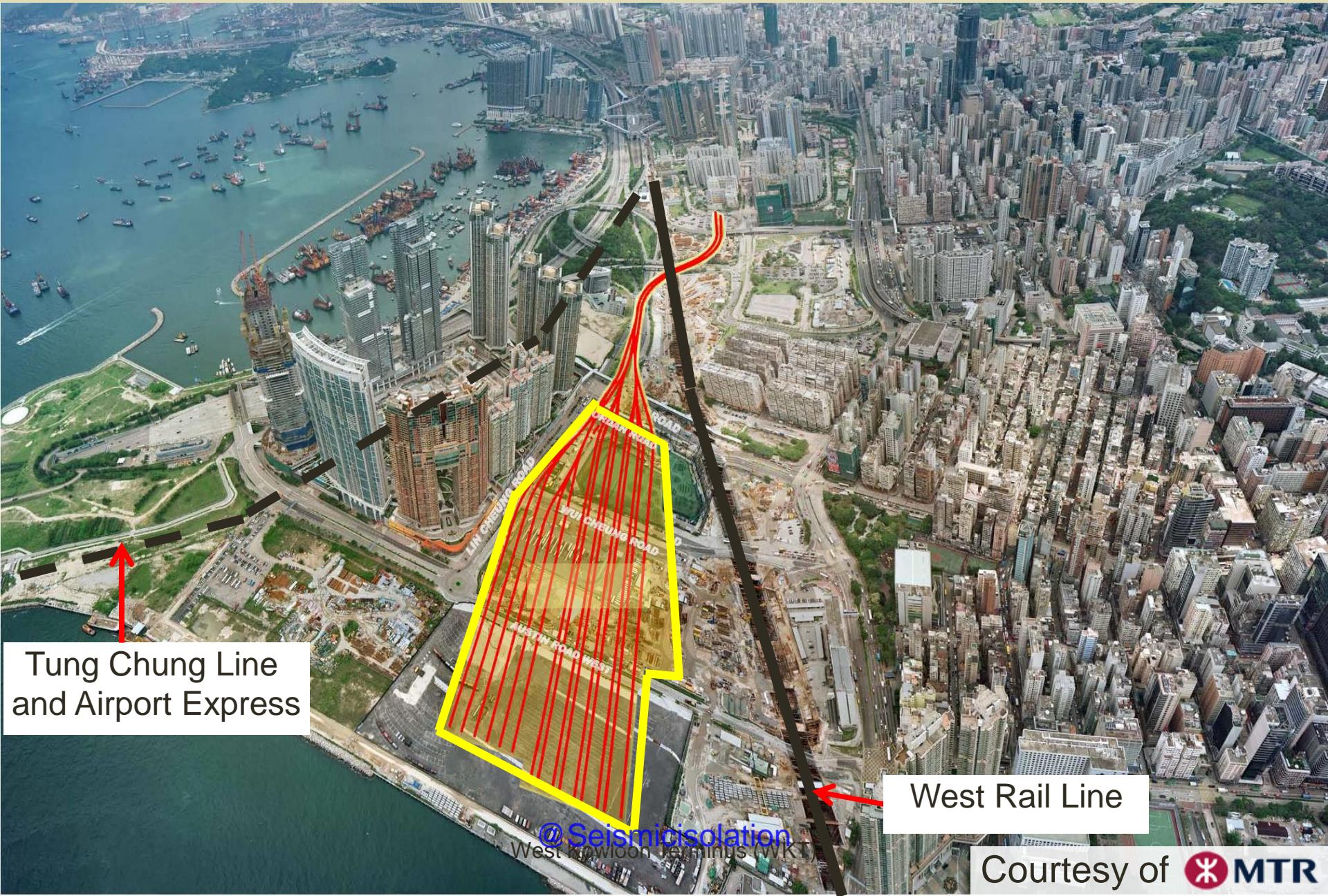
Underpinning ISL Finger Platform Tunnel

Break Through



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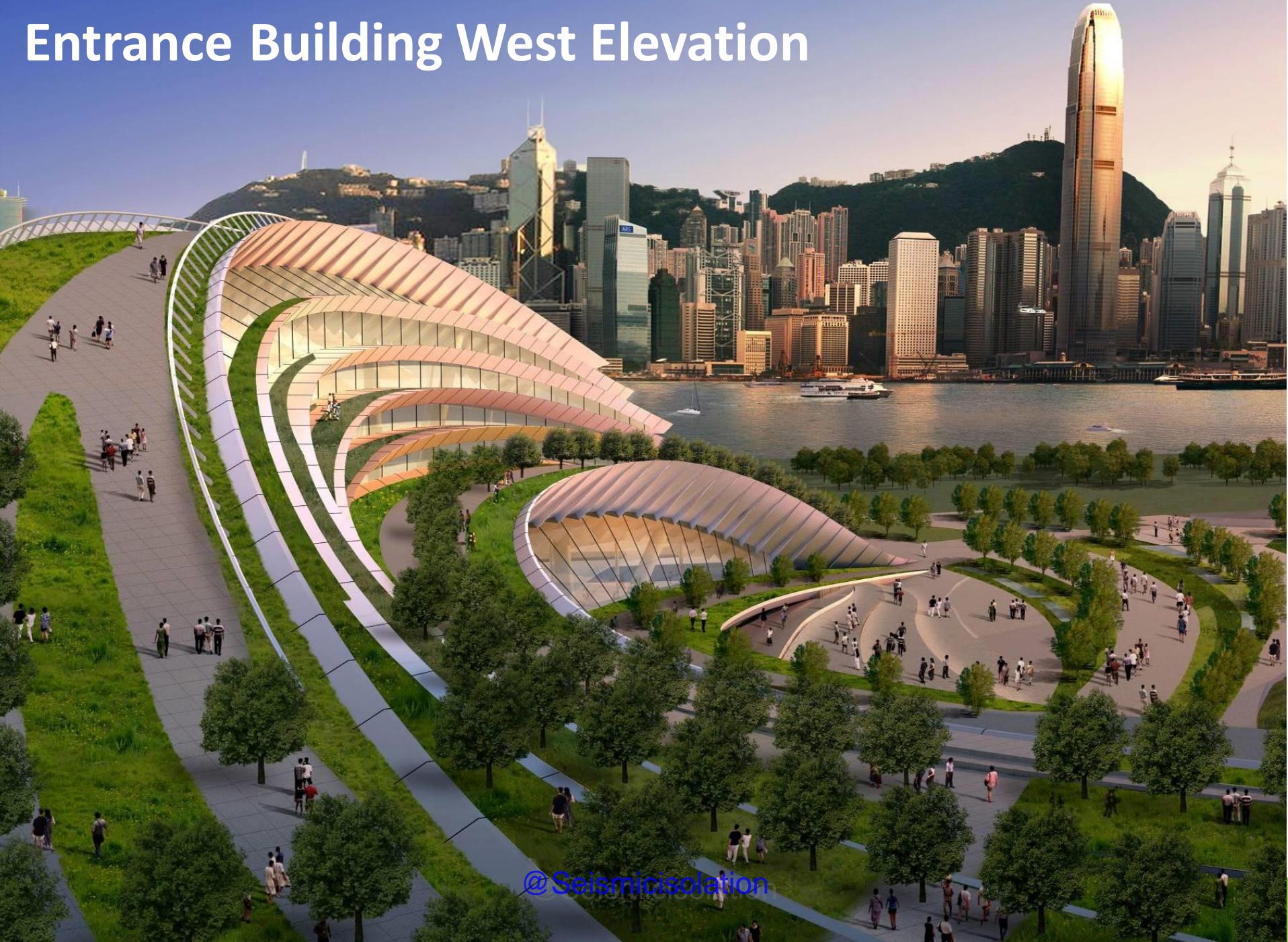
Case 3: West Kowloon High Speed Railway Terminus



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West Kowloon Terminus (WKT)

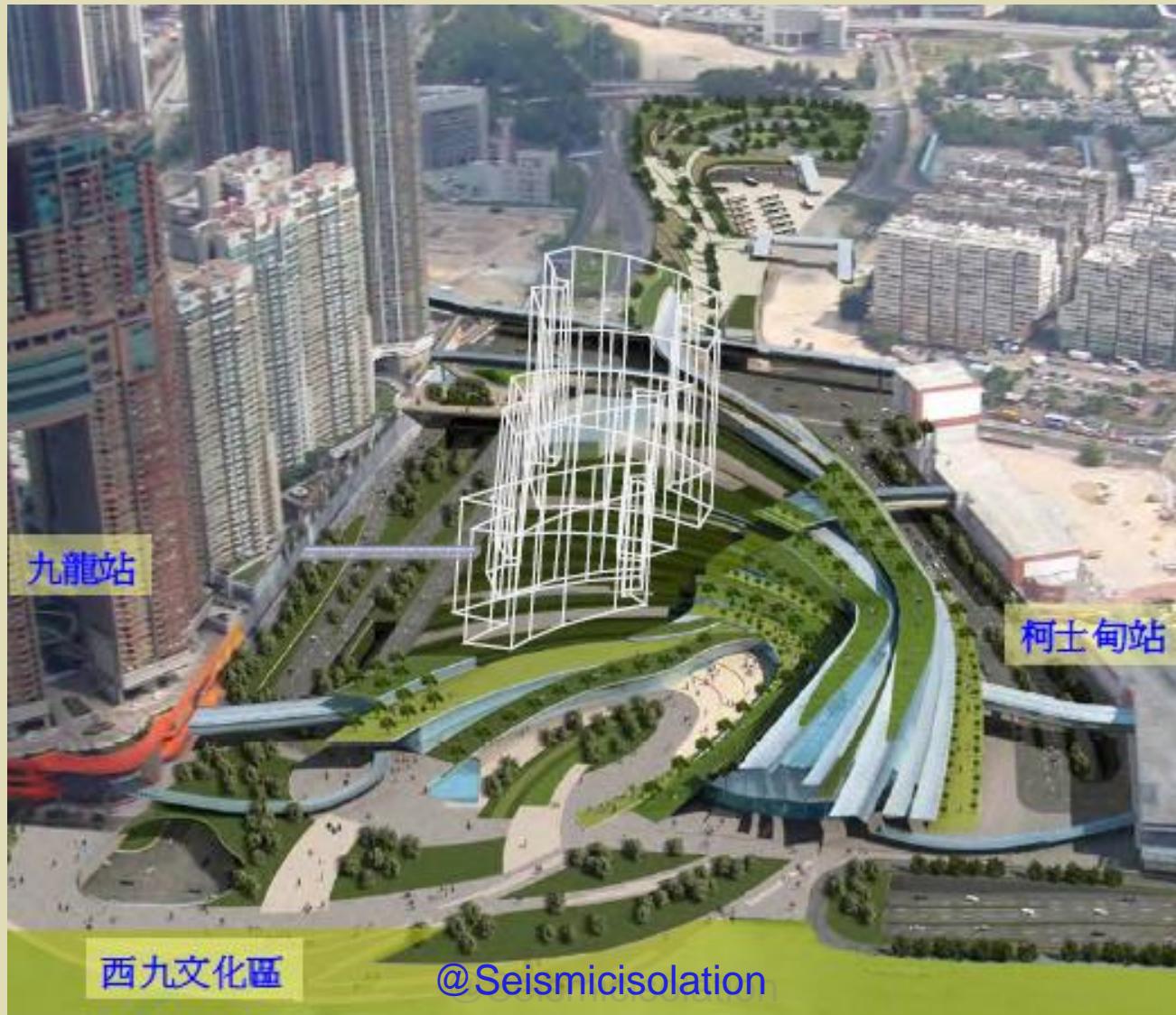
Courtesy of  MTR

Entrance Building West Elevation



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Planned Co-development





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Reference Scheme

The technical challenge:
a deep and extensive
excavation in an
intensively developed
district.

11 hectares of site area
(1 million square feet)



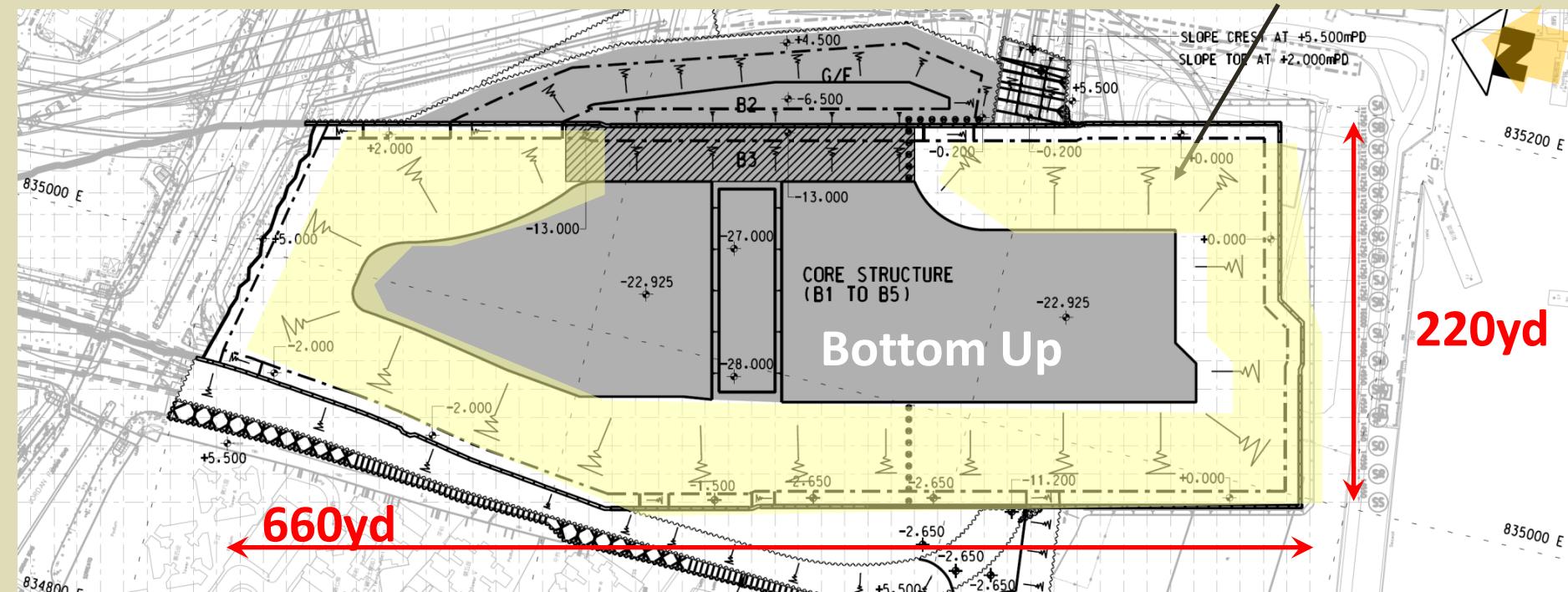
3.5 M cubic yd. excavation volume



Concept: Big Dig and Bottom-up in the middle then Top-down around the perimeter.

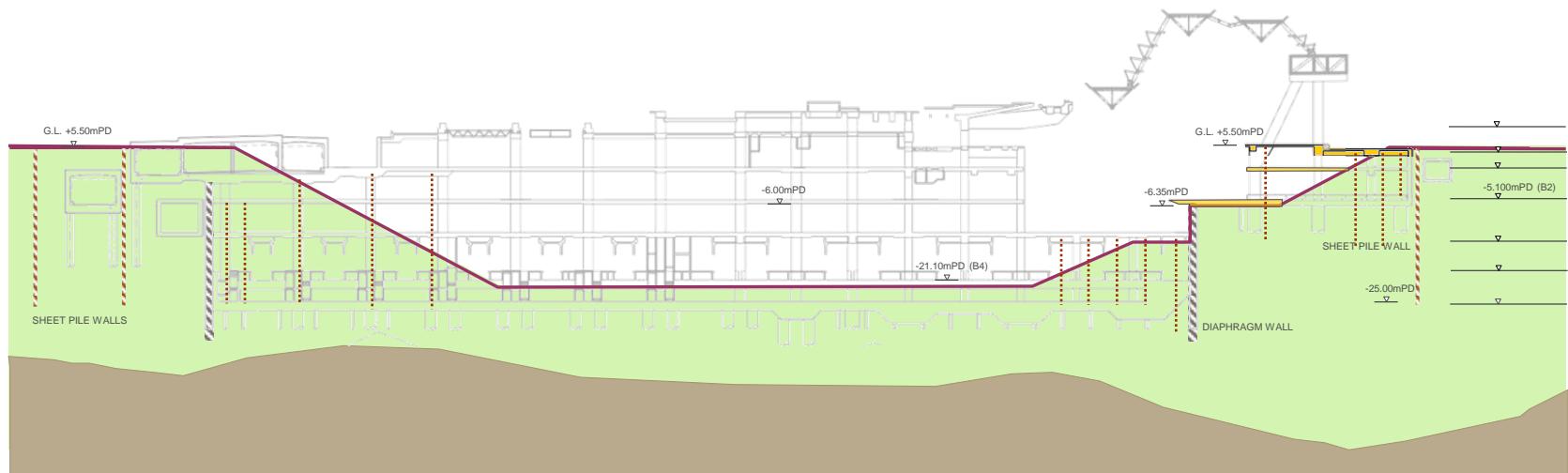
- 1 – Diaphragm Walls and Foundations
 - 2 – Rapid Open Bulk Excavation with Temporary cut slopes
 - 3 – Bottom-up construction of the central core
 - 4 – Remove slopes with Top-down construction at the perimeter

Top Down

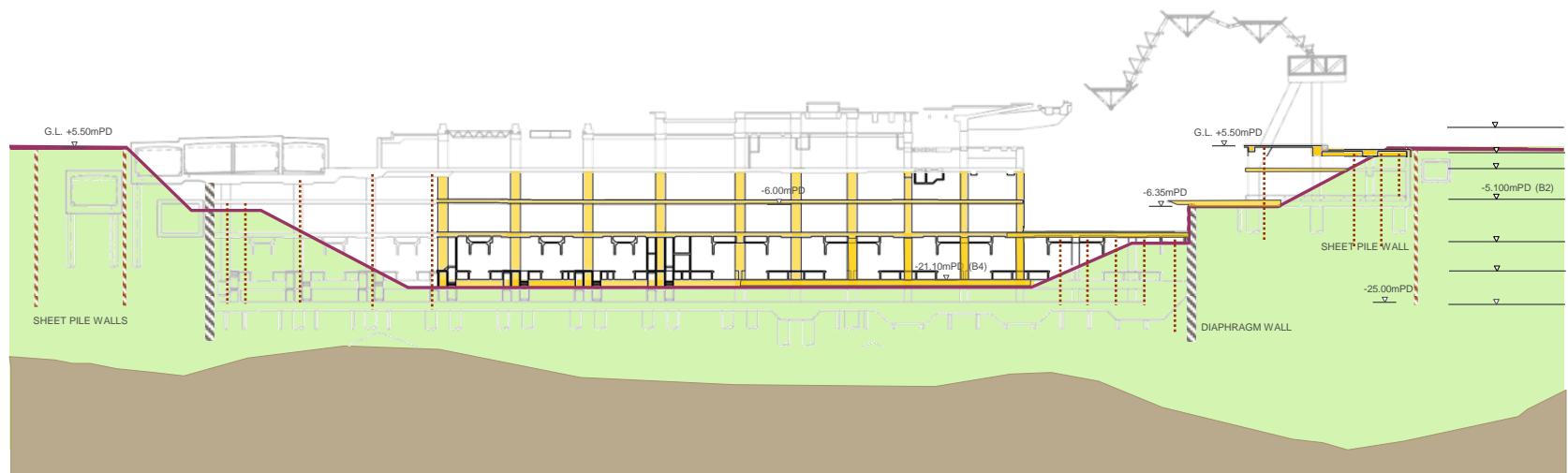


Excavate to -21mPD (B4)

Construct B1 and B2



Bottom-up construction in the middle



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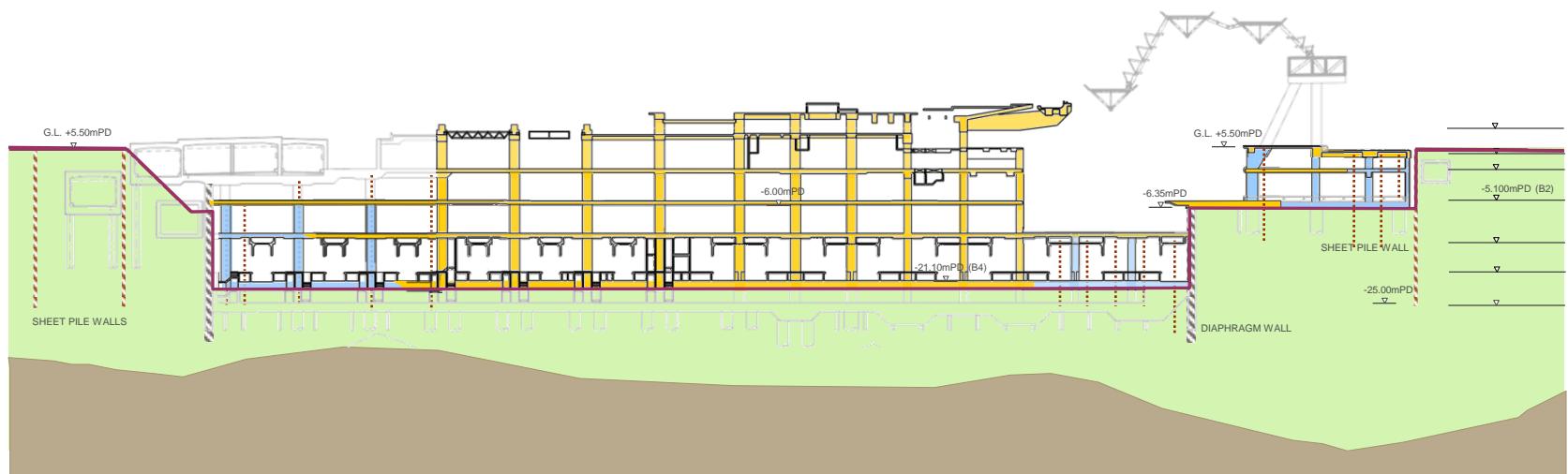
Construct B3, B2 in the middle



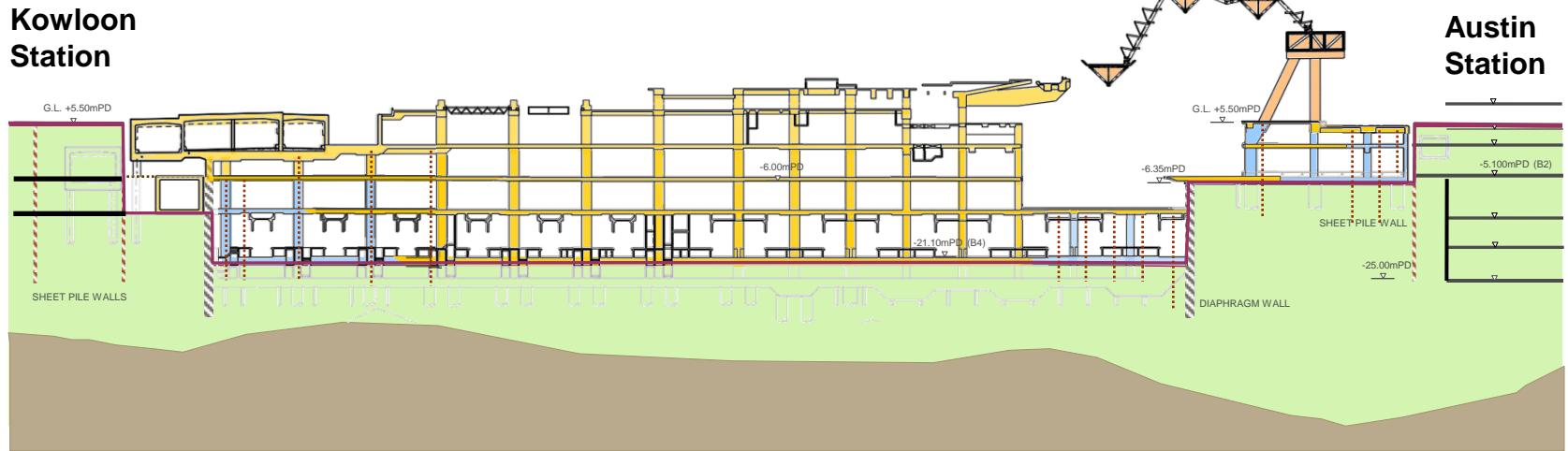
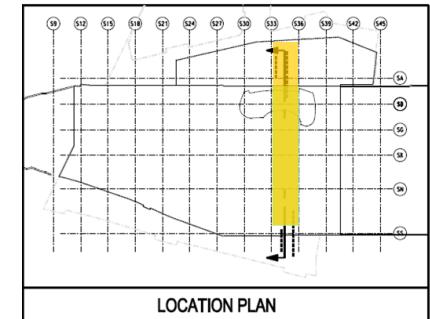
@Seismicisolation

Partial Top-down on eastern and western sides

Continue Bottom-up B1 and G/F on middle part



Construct western underpass and roof structures

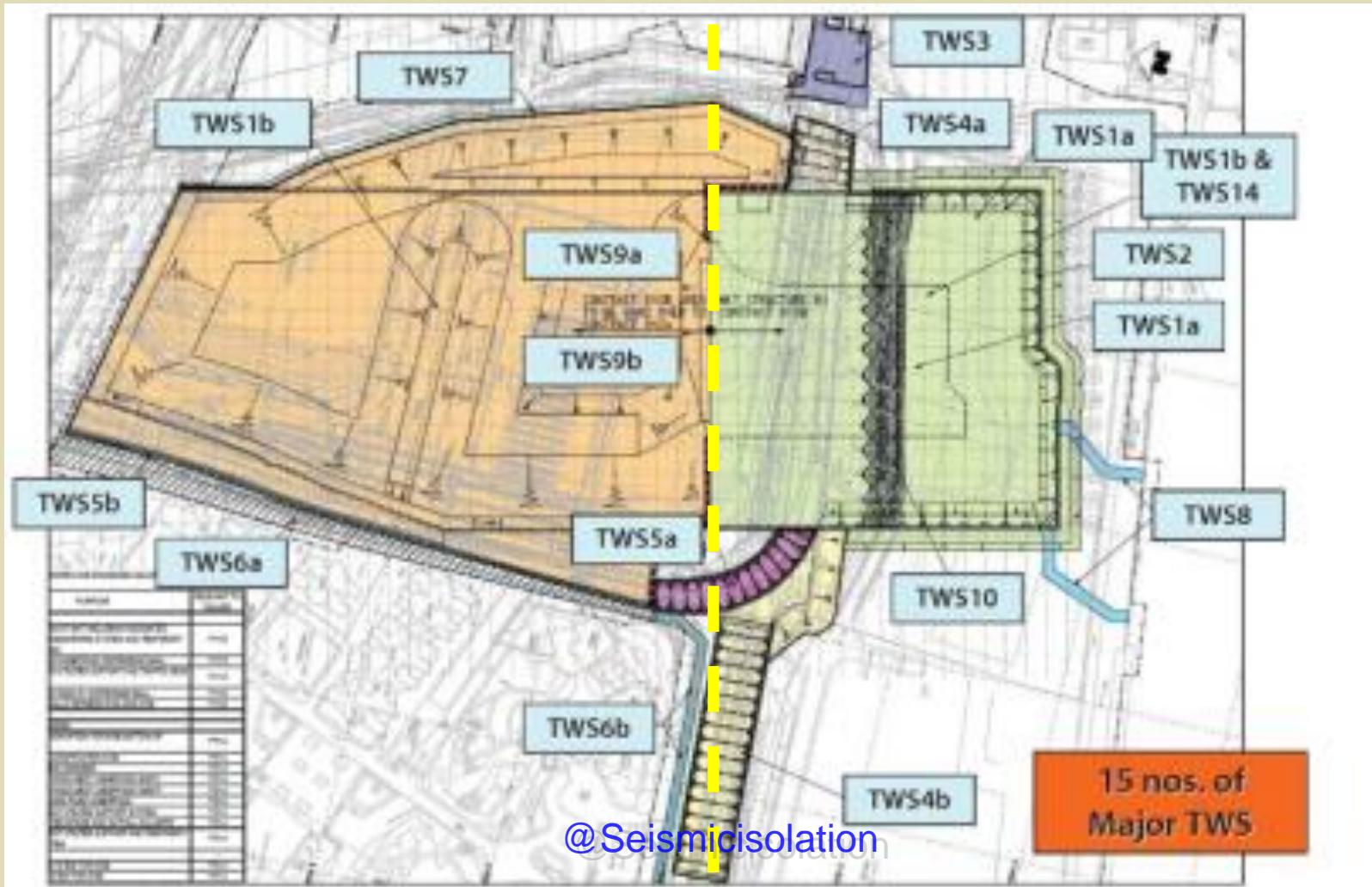


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Separate Contracts :

Reference Scheme
with shared haul road

All Top-down



Deep Excavation at West Kowloon High Speed Rail Terminus

Excavation for Bottom-up

G/f construction

Top-down



Later stage of construction

Bottom-up structure linking to Top-down



Top-down Bottom-up construction below ground



Ground level

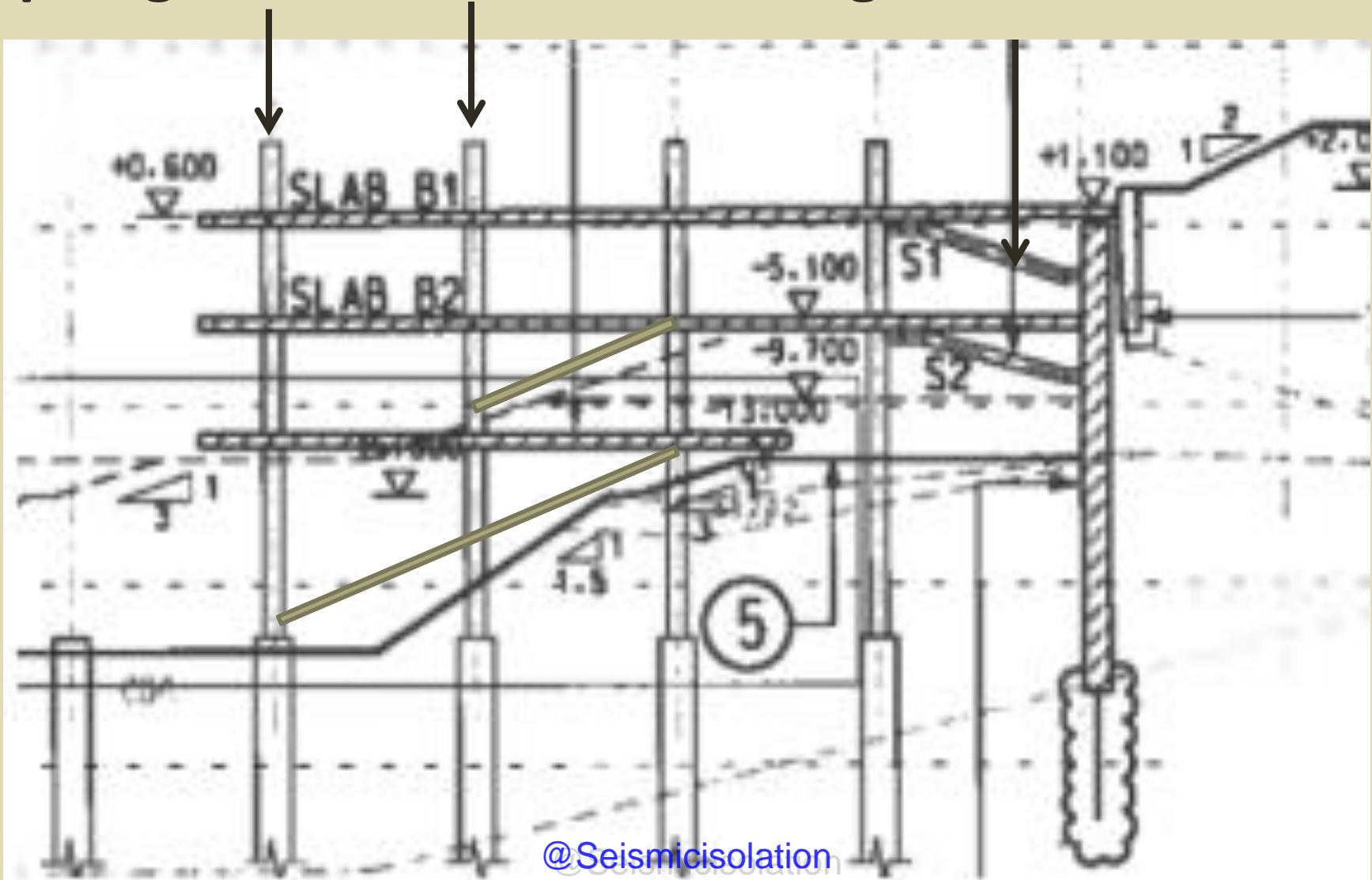


30m deep

Shared haul road



Top-down: plunged columns and raking shores



Top-down:

Plunged steel columns to be encased as permanent columns



Top-down:

Permanent columns cast around plunged columns
and raking shores



Bottom-up: In situ columns with steel cores for capacity



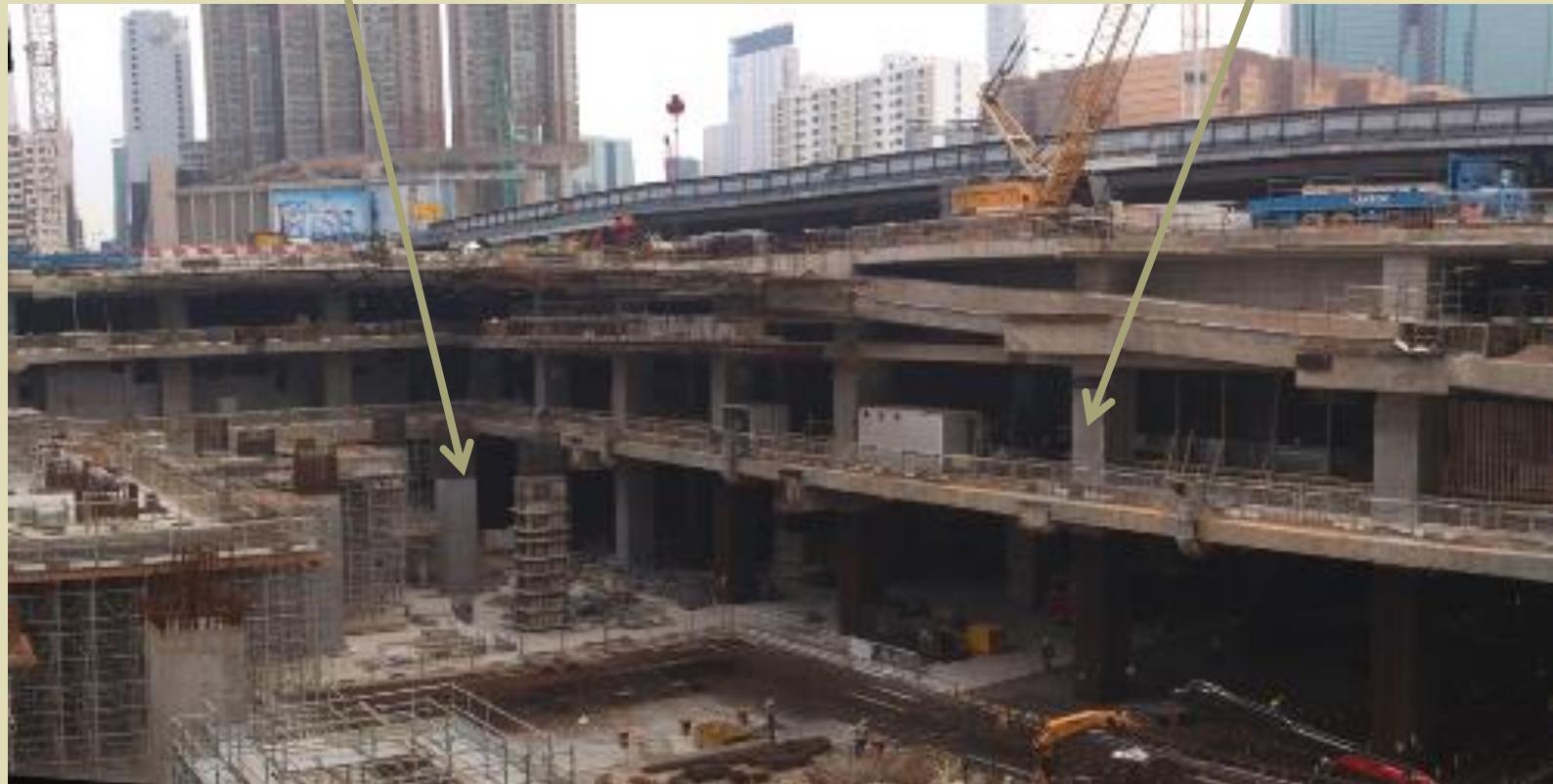
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Bottom-up: Steel core for heavy load columns for future buildings.



Bottom-up
in-situ columns

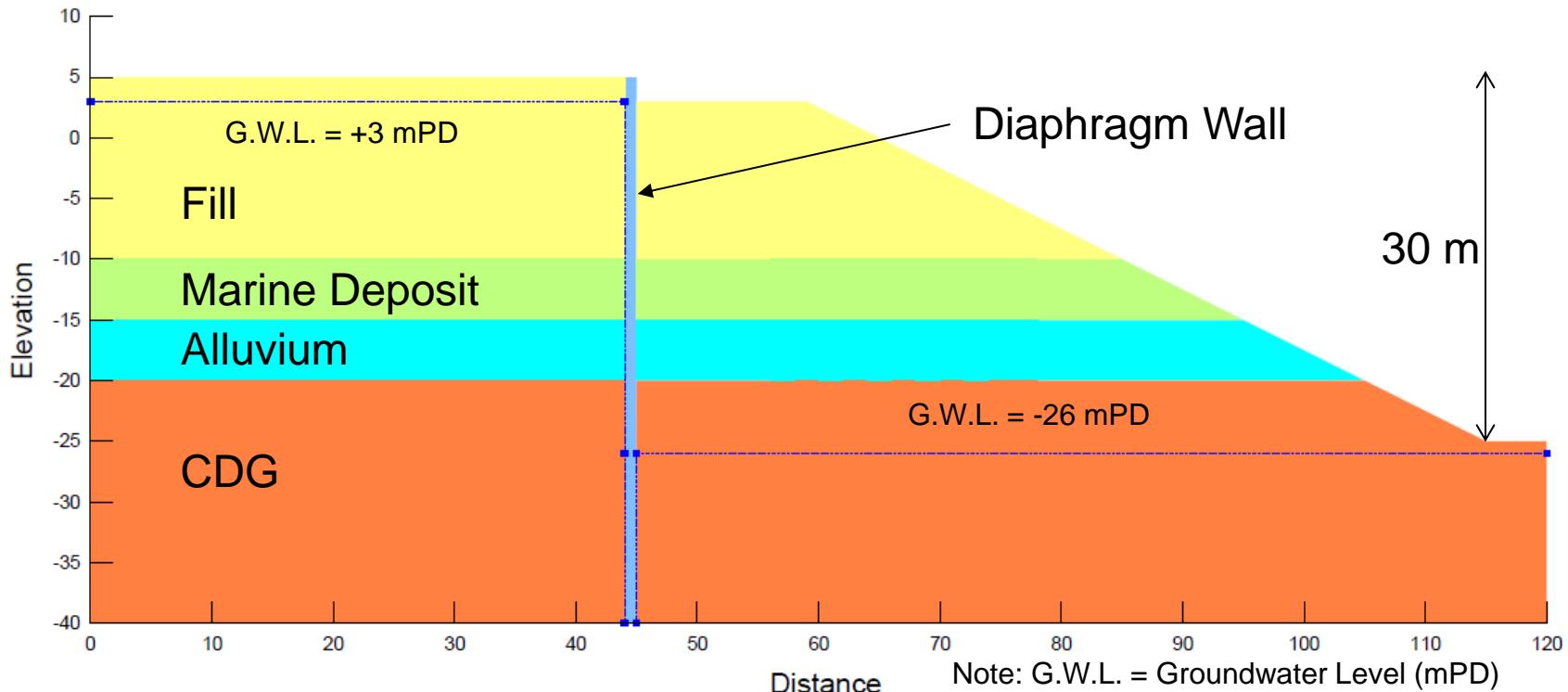
Top-down
Encased plunged columns



Unusual numerical modelling

1. Stability of temporary side slopes
 - Open cut excavation supported by temporary slopes (finite element vs limit equilibrium)
 - Modelling of ground treatment to enhance overall stability
2. 3-D modelling of adjacent diaphragm wall movements
 - Different construction sequences may induce 3-D wall movements
 - Modelling of panel connections

Slope Stability Assessment – Limit Equilibrium Method



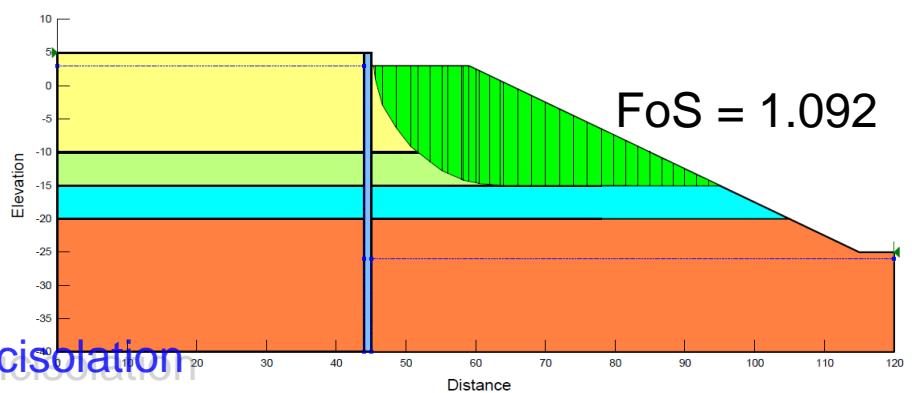
Input Parameters:

<u>Fill</u>	<u>Marine Deposit</u>	<u>Alluvium</u>
$\gamma = 19 \text{ kN/m}^3$	$\gamma = 17 \text{ kN/m}^3$	$\gamma = 19.5 \text{ kN/m}^3$
$c = 0 \text{ kPa}$	$c_u = 40 \text{ kPa}$	$c = 0 \text{ kPa}$
$\phi = 36^\circ$		$\phi = 36^\circ$

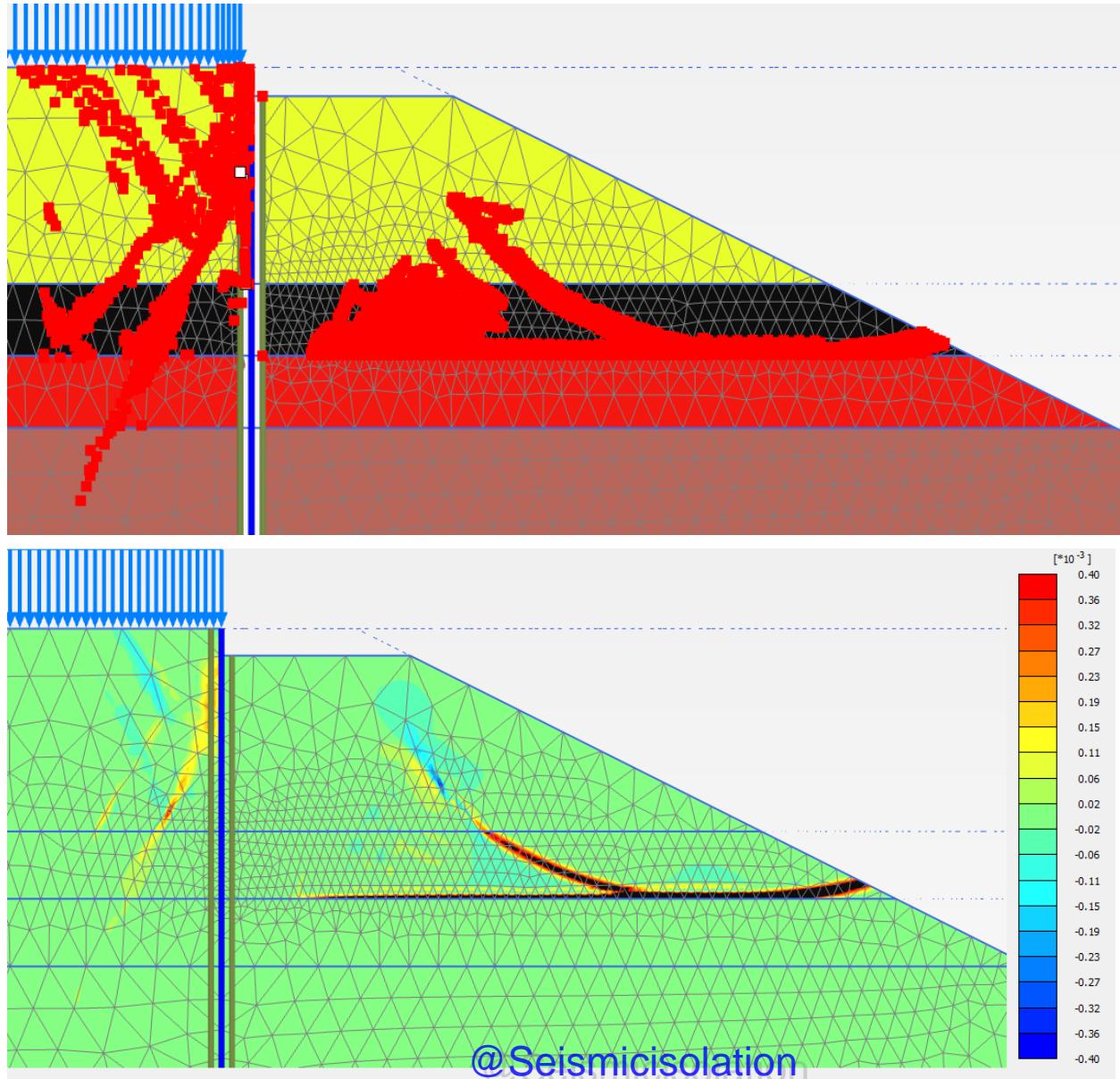
CDG

$\gamma = 19 \text{ kN/m}^3$
$c = 5 \text{ kPa}$
$\phi = 35^\circ$

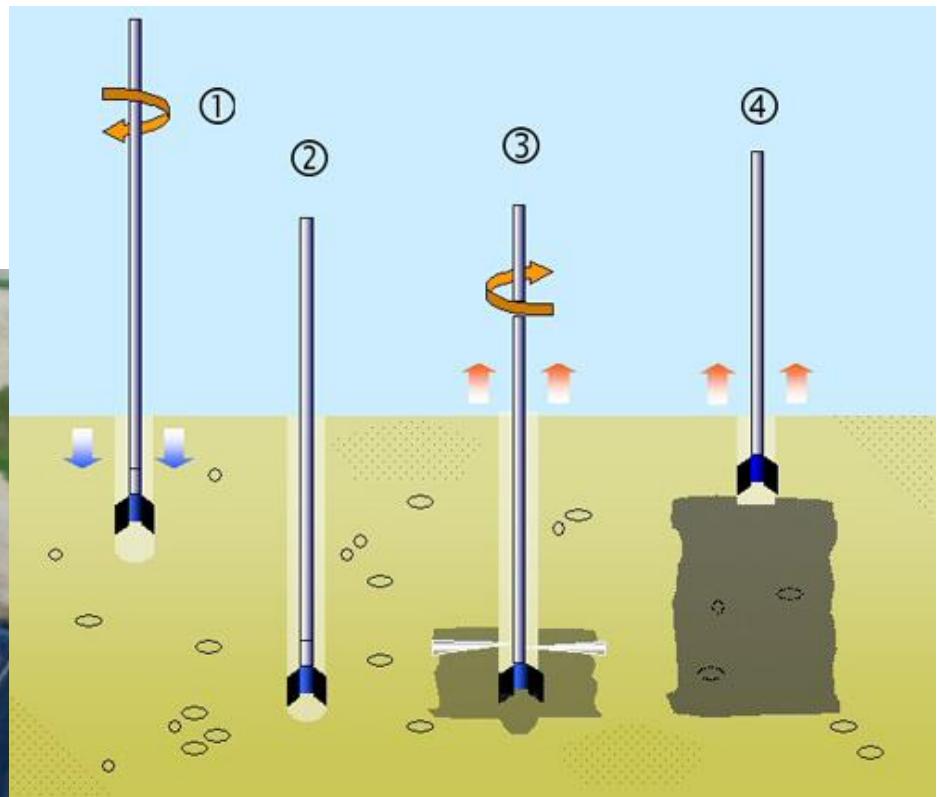
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Finite Element Modelling – Plaxis



Ground Treatment – Jet Grouting

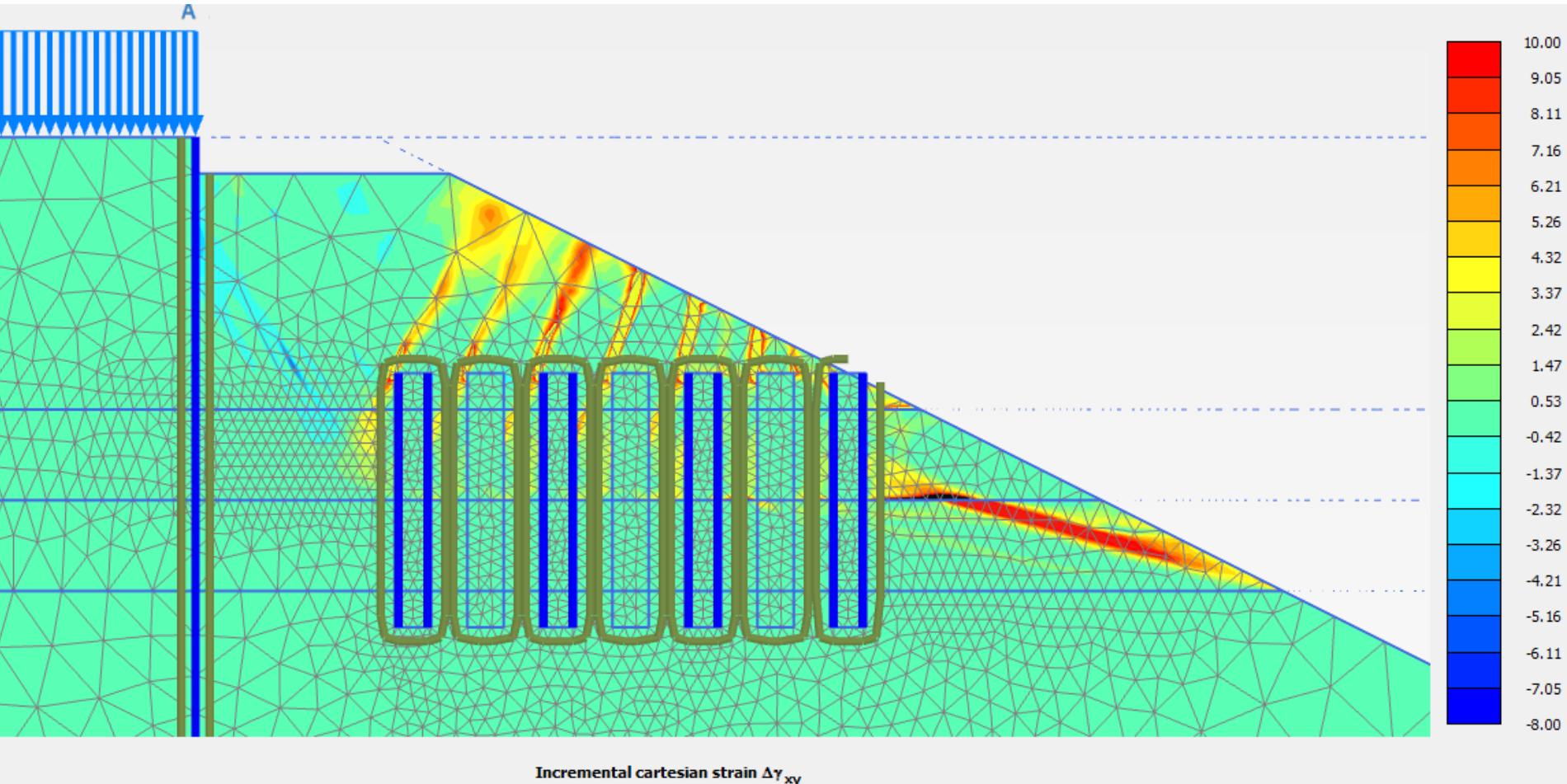


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Finite Element Modelling of Jet Grouting

xy strain



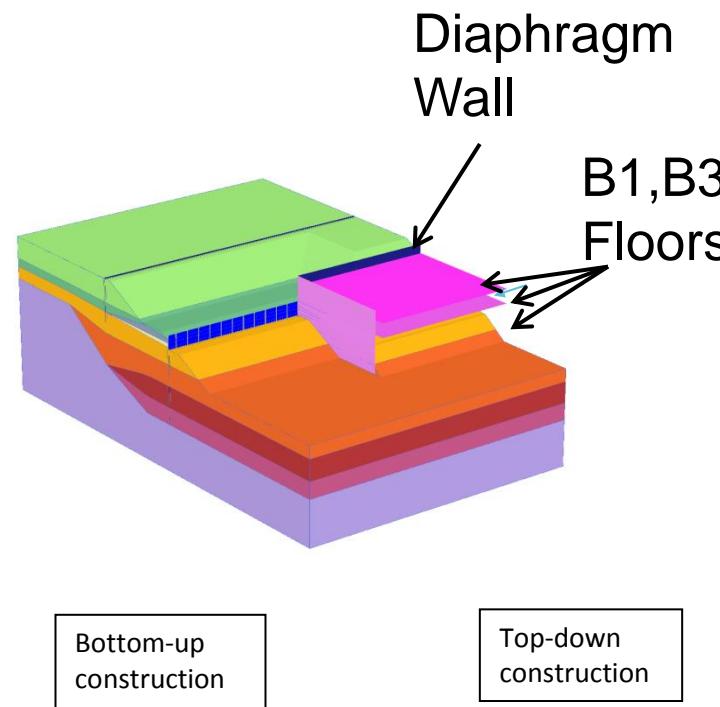
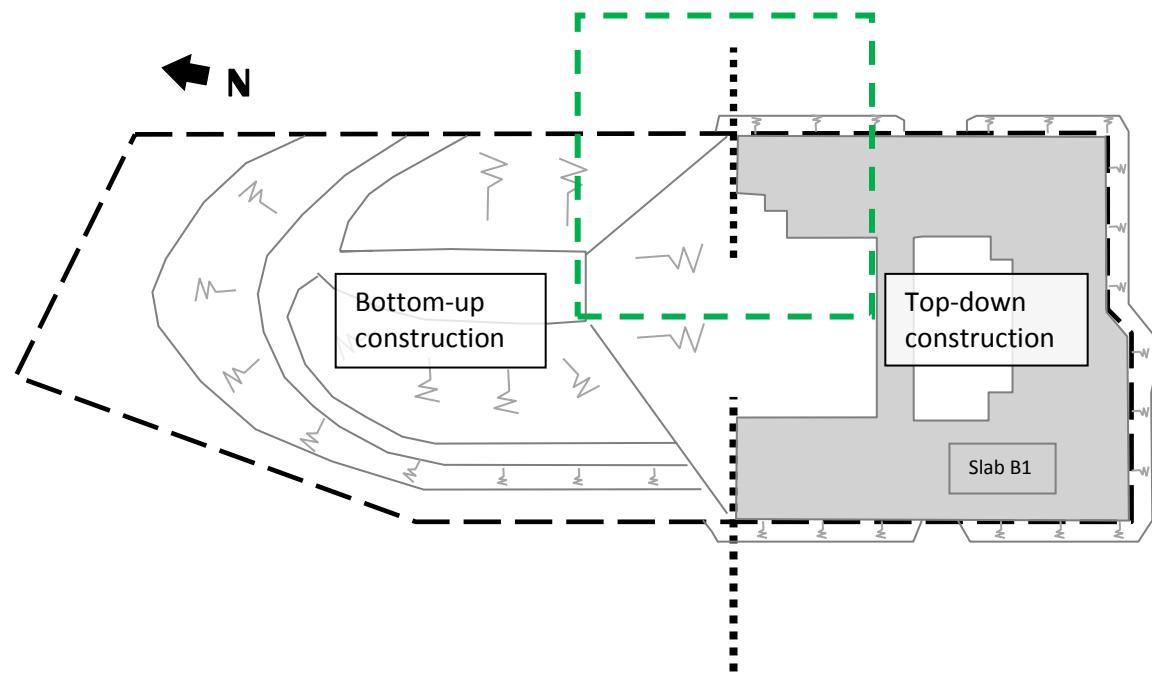
Slip elements between JGP columns

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Bottom-up/Top-down interface

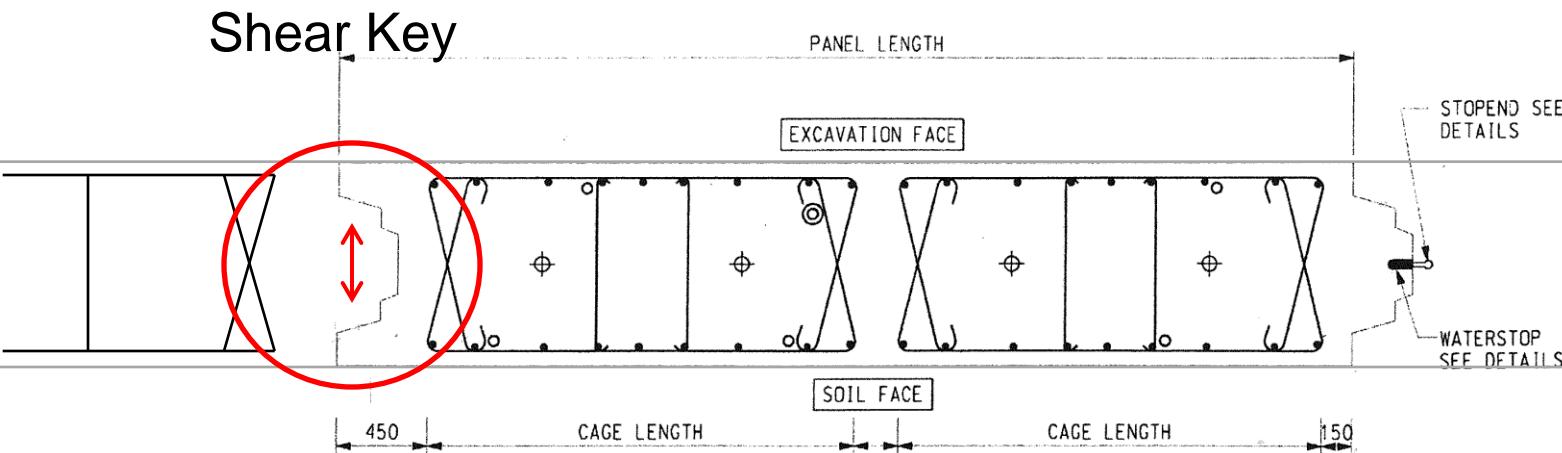
East Side 3D Model



Details of Actual Panel-Panel Connections

No reinforcement across the shear key :-

- Bending moments cannot be transferred between D-wall panels
- Shear forces can be transferred by plain concrete

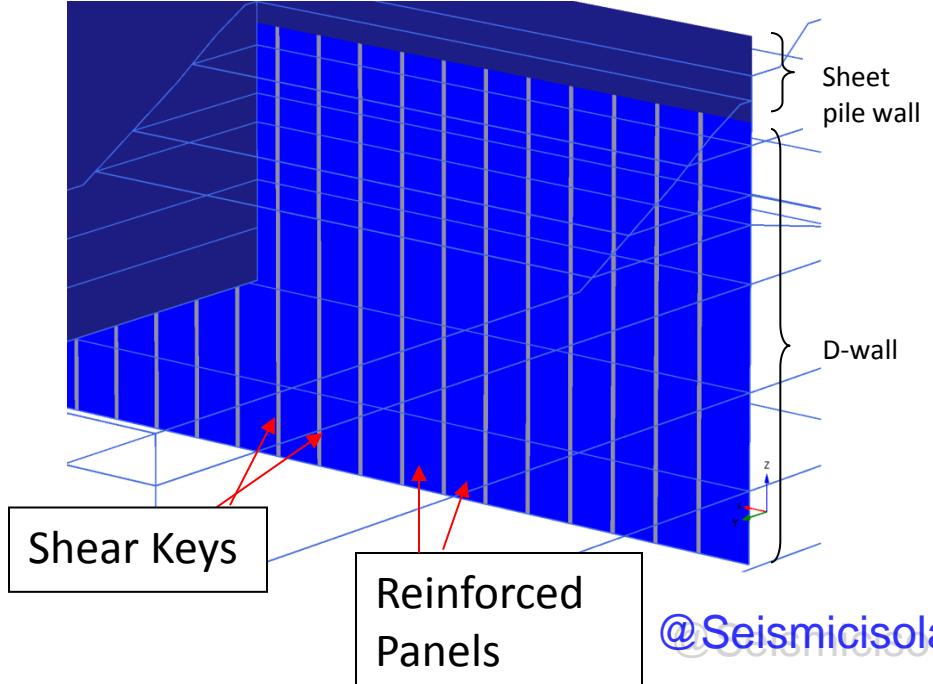


Can rotate = no bending moment

Resists shear

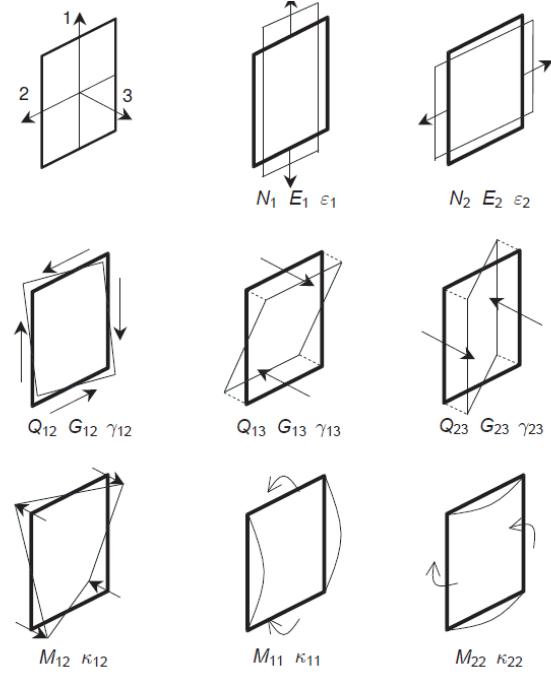
Modelling of Diaphragm Wall

- Isotropic reinforced concrete properties for the reinforced blue panels
- Anisotropic properties for grey shear keys, with E_2 (Young's modulus in lateral direction) close to zero, and shear modulus unchanged in both directions



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$$\begin{aligned} \begin{bmatrix} N_1 \\ N_2 \end{bmatrix} &= \begin{bmatrix} E_1 d & \nu_{12} E_2 d \\ \nu_{12} E_2 d & E_2 d \end{bmatrix} \begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \end{bmatrix} \\ \begin{bmatrix} Q_{12} \\ Q_{13} \\ Q_{23} \end{bmatrix} &= \begin{bmatrix} G_{12} d & 0 & 0 \\ 0 & kG_{13} d & 0 \\ 0 & 0 & kG_{23} d \end{bmatrix} \begin{bmatrix} \gamma_{12} \\ \gamma_{13}^* \\ \gamma_{23}^* \end{bmatrix} \\ \begin{bmatrix} M_{11} \\ M_{22} \\ M_{12} \end{bmatrix} &= \begin{bmatrix} \frac{E_1 d^3}{12} & \frac{\nu_{12} E_2 d^3}{12} & 0 \\ \frac{\nu_{12} E_2 d^3}{12} & \frac{E_2 d^3}{12} & 0 \\ 0 & 0 & \frac{G_{12} d^3}{12} \end{bmatrix} \begin{bmatrix} \kappa_{11} \\ \kappa_{22} \\ \kappa_{12} \end{bmatrix} \end{aligned}$$

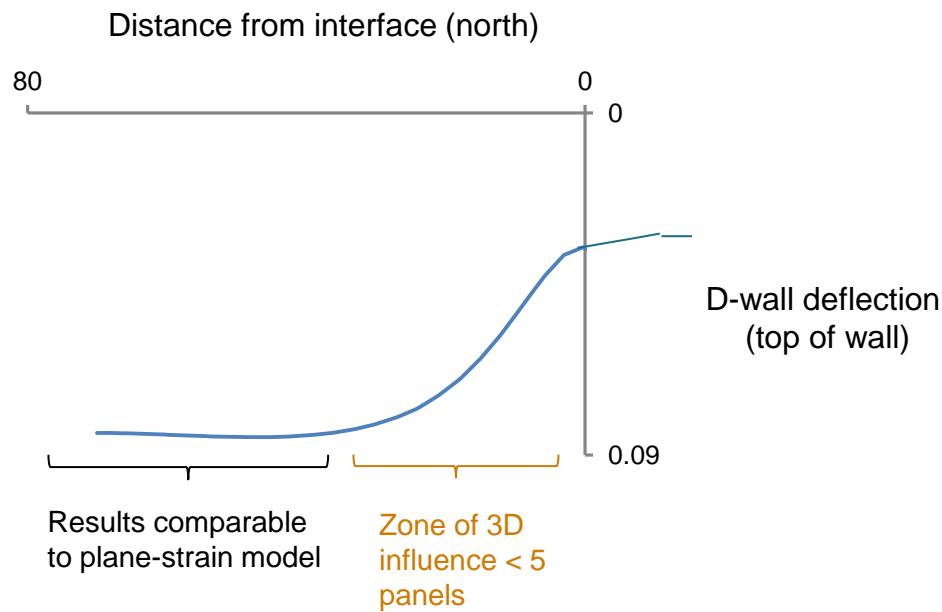


(PLAXIS 3D manual)



Lateral Influence Zone

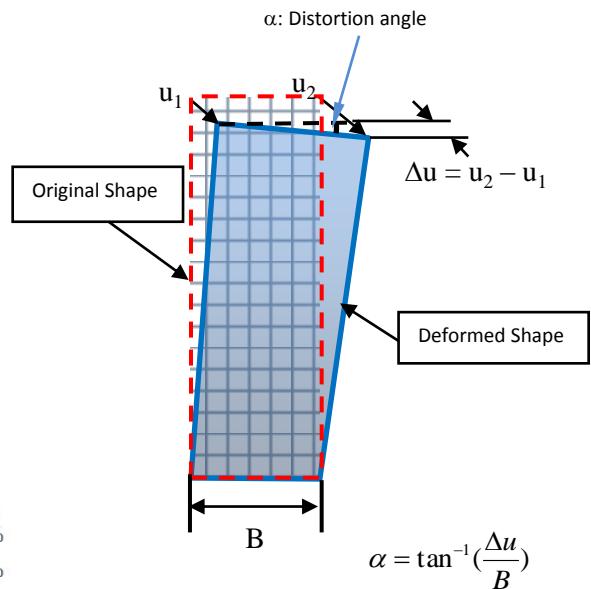
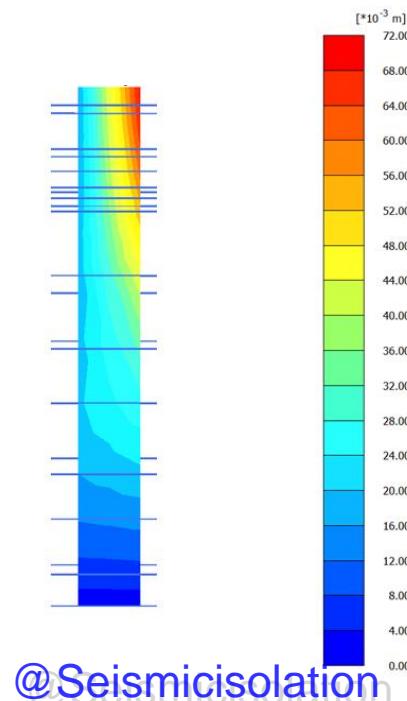
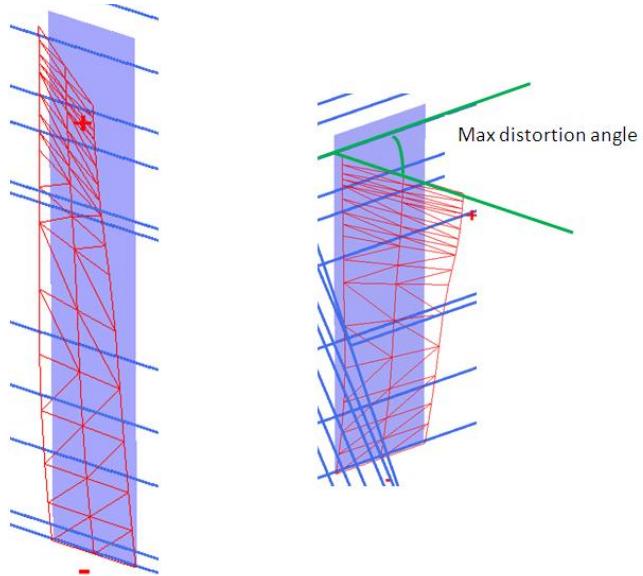
- Bending moments not transferred between panels, so they rotate freely with respect to each other
- Shear forces are transferred between panels (G_{23} unchanged), mimicking the function of shear key
- Axial forces in lateral direction cannot be transferred between panels
- 3-D modelling allows the potential influence zone to be identified.
- Outside the influence zone, wall deflection is comparable to 2-D estimates



3-D Movement Pattern of the Diaphragm Wall

- Difference in construction sequences can lead to distortion of D-wall panels near the interface, which cannot be captured in 2-D models
- Modelling of individual panels allows evaluation of this distortion for each panel

Deformed Shape (exaggerated)



New Topic:

Back Analyses

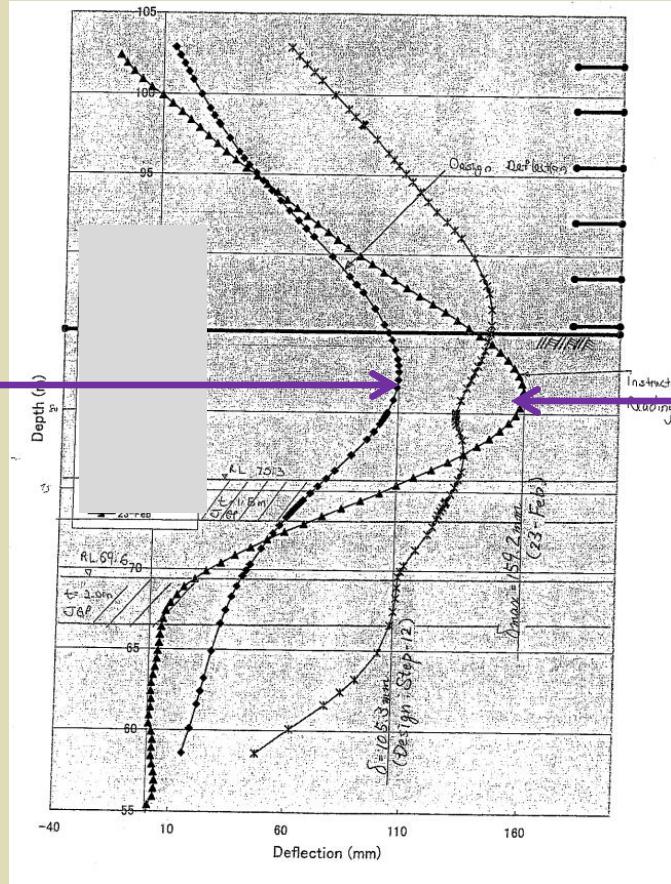
Fundamental to “Observational Method”

Useful for understanding how the ground and structures are behaving

At a given stage of construction a “Stop Work” limit was exceeded

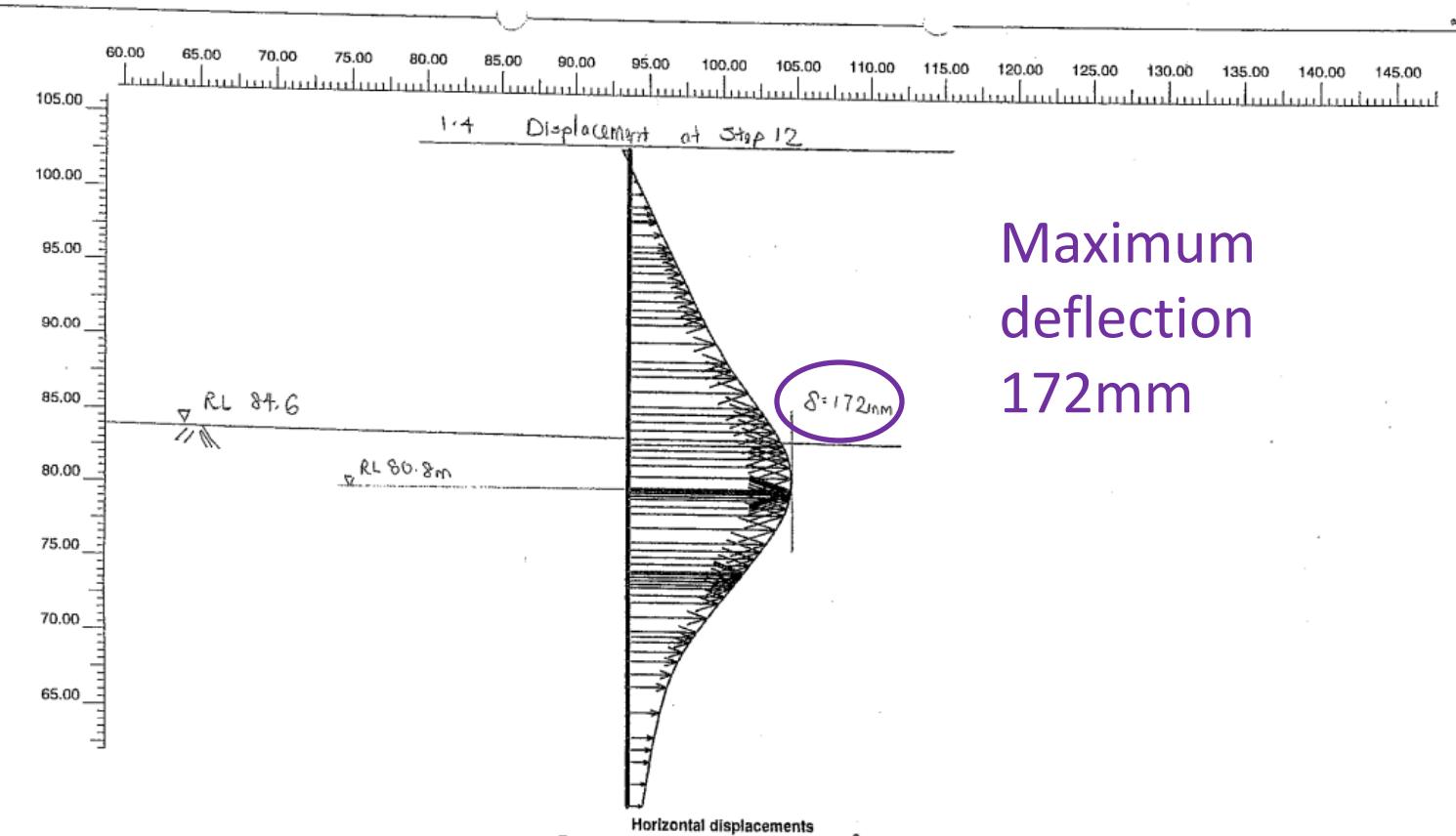
Designer's prediction at this stage
105.3mm

Monitored Maximum deflection 159.2mm



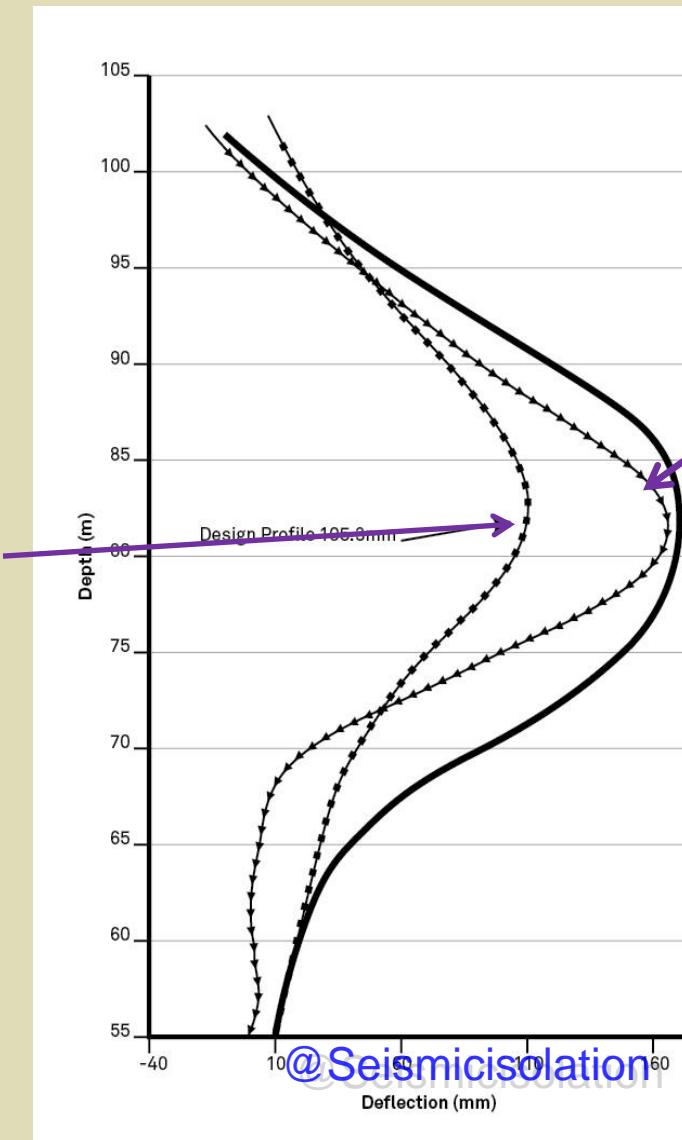
Inclinometer at cut and cover tunnel excavation, Singapore

Back analysis for this stage :



Back analysis showed wall was within limits so the Stop Work limit was increased to 300mm

Design
Profile
105.3mm



Monitoring
159.2mm

Back analysis
172mm

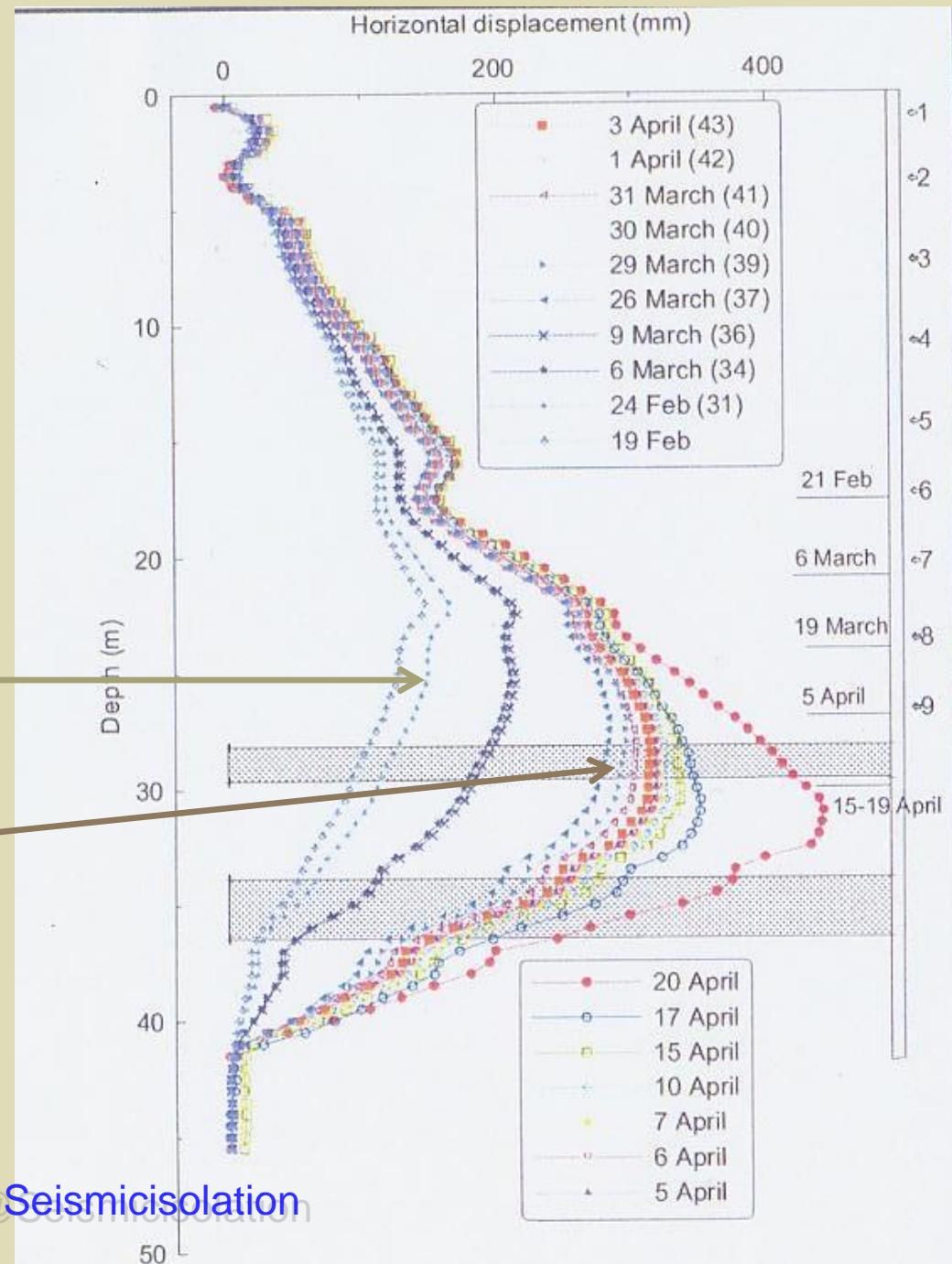
Bending moment = "EI"/Radius

Later the wall moved
more than 300mm
A second back analysis
was required

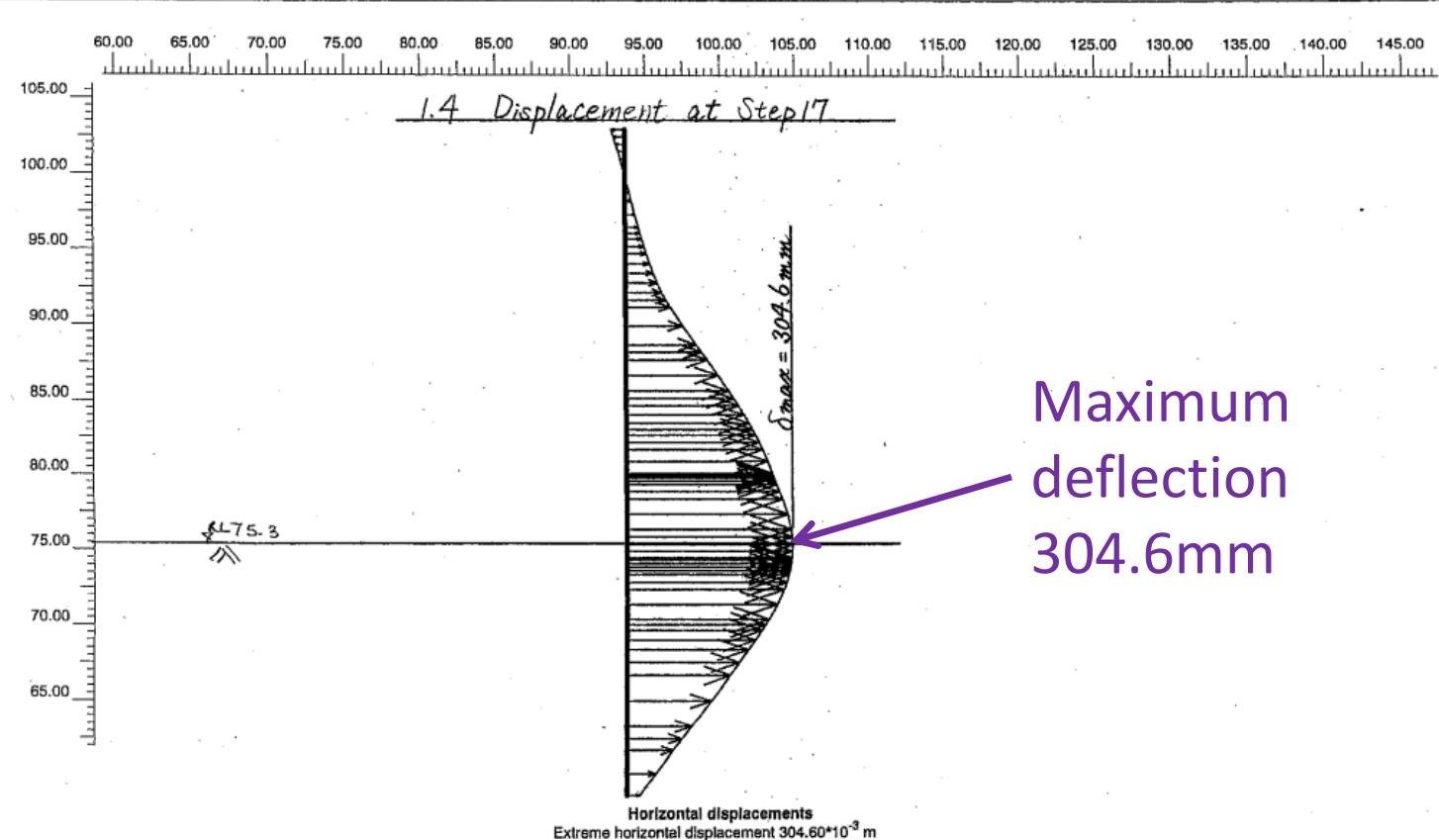
First back analysis 23 February
159.2mm

Second back analysis 3 April
300mm

Work continued Deflection
reached 400mm 20 April



Second Back Analysis



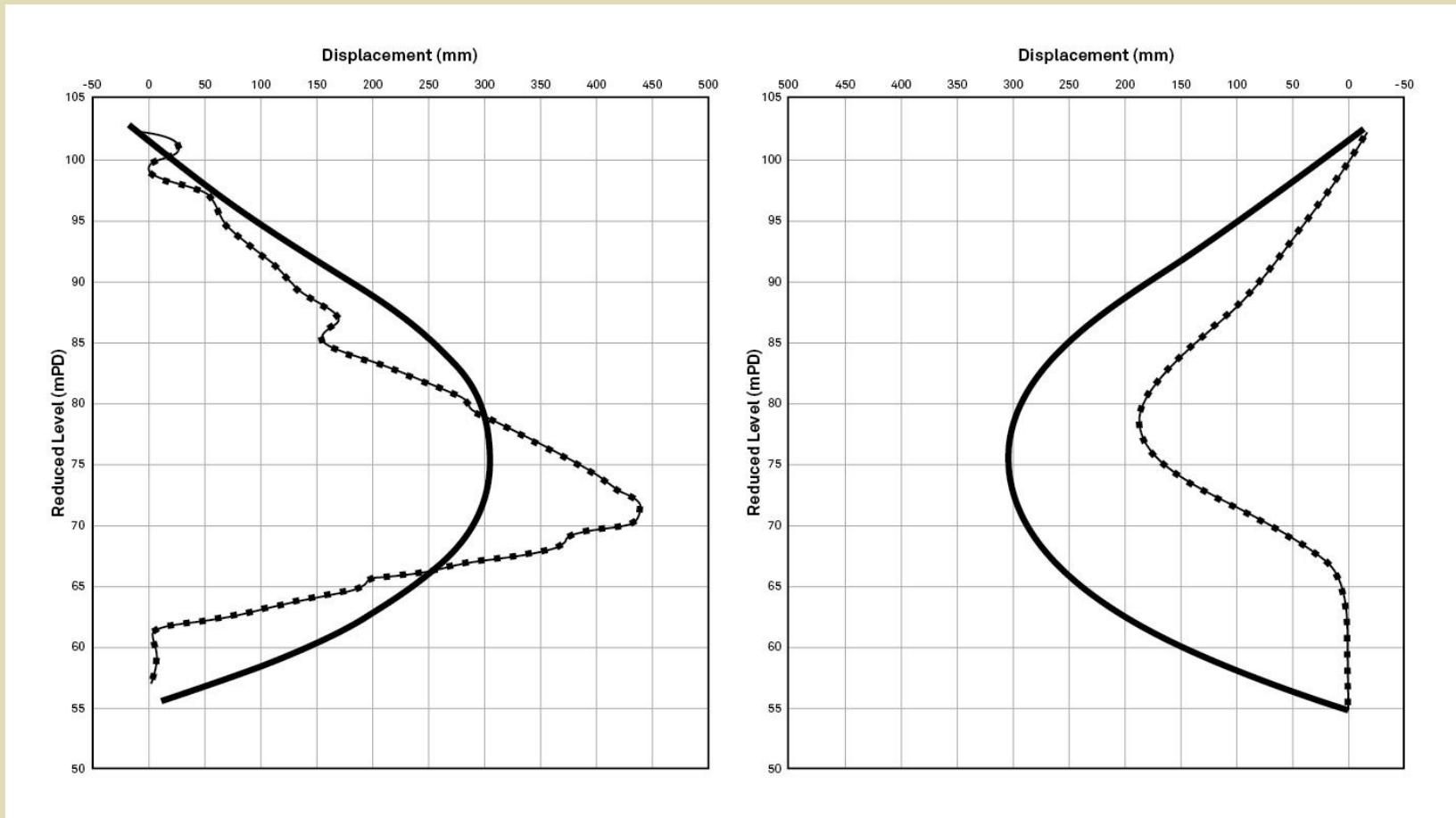
PLAXIS
Finite Element Code for Soil and Rock Analyses

Version 7.2.10.149

Project description		Type-M3		
Project name	Step	200	Date	User
TYPE-M3	200	04/05/04		

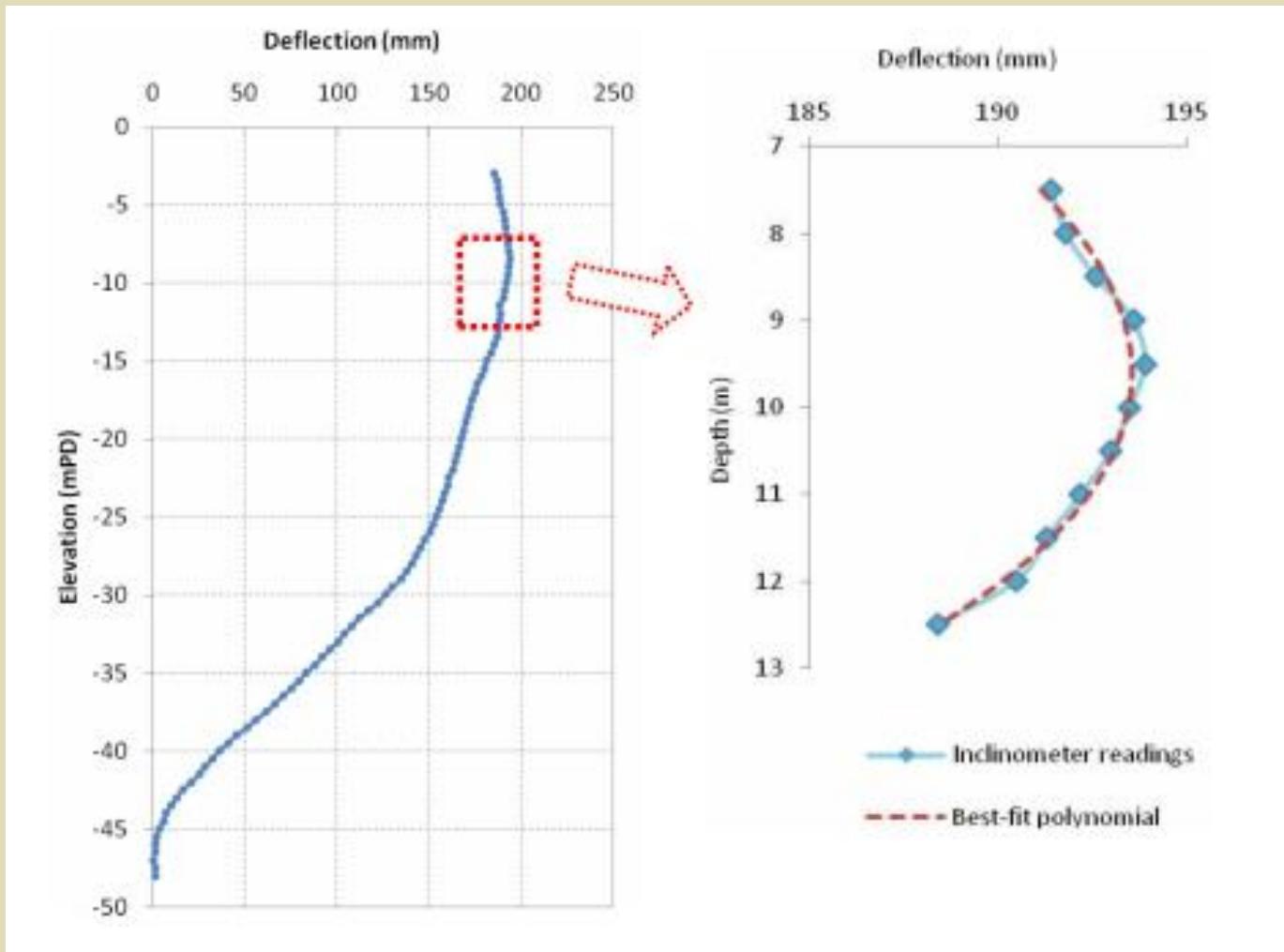
07

The back analysis profile did not fit either inclinometer profile



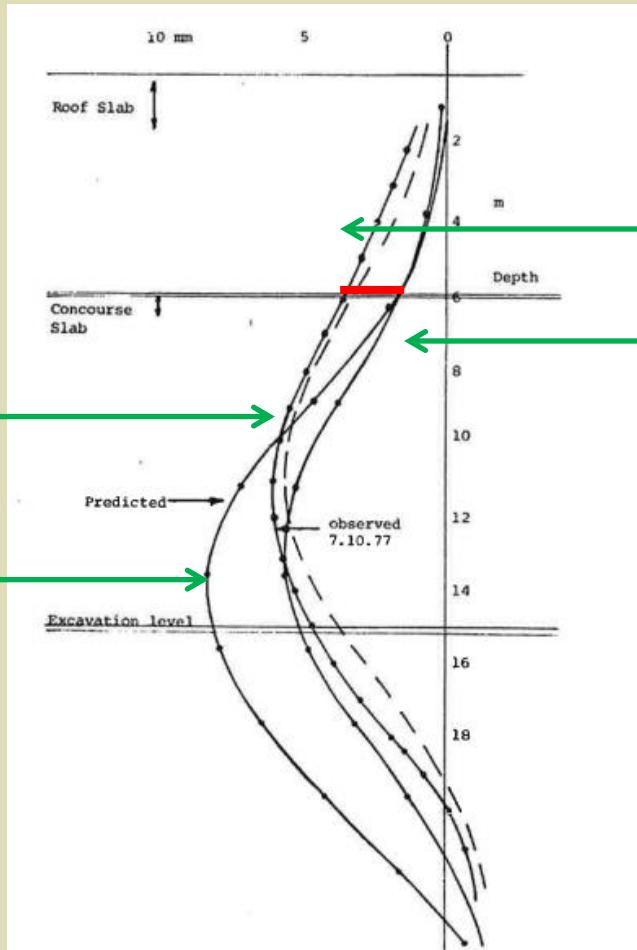
Bending moment = "EI"/Radius
© Seismicisolation

Back analyses of deflections: Fourth order difference



Another project

Back analysis at pre-final stage allowed deletion of lowest strut



Lessons:-

1. High shrinkage of concrete resulted in twice estimated compression of concourse slab
2. Lateral earth pressure less than estimated and reduced by de-watering.

Observational Method
allowed omission of
lowest strut

Note the headroom

Plunged permanent RC
columns supported by
steel stools for connection
to base slab.



Case 3

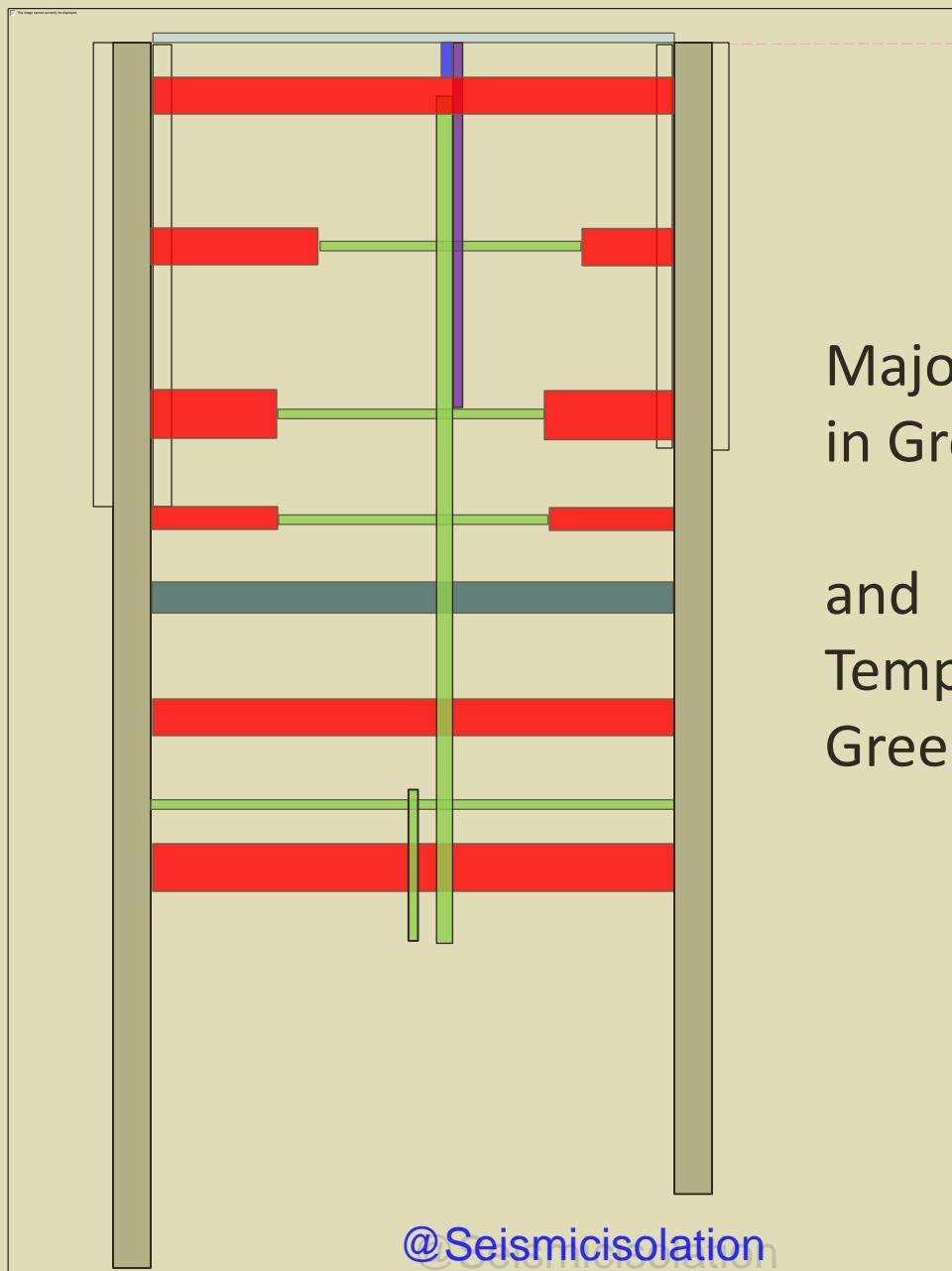
Mixture, top-down, bottom up, attached shaft



Case 4: Fashionable to have open ceilings



EXISTING GROUND LEVEL 103.50



Major structural elements
in Grey and Red
and
Temporary strutting in
Green and Purple

Heavy structures around openings



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Topic 2: Plunge Piles

These are ‘plunged’ into the ground at the beginning in order to support top-down construction

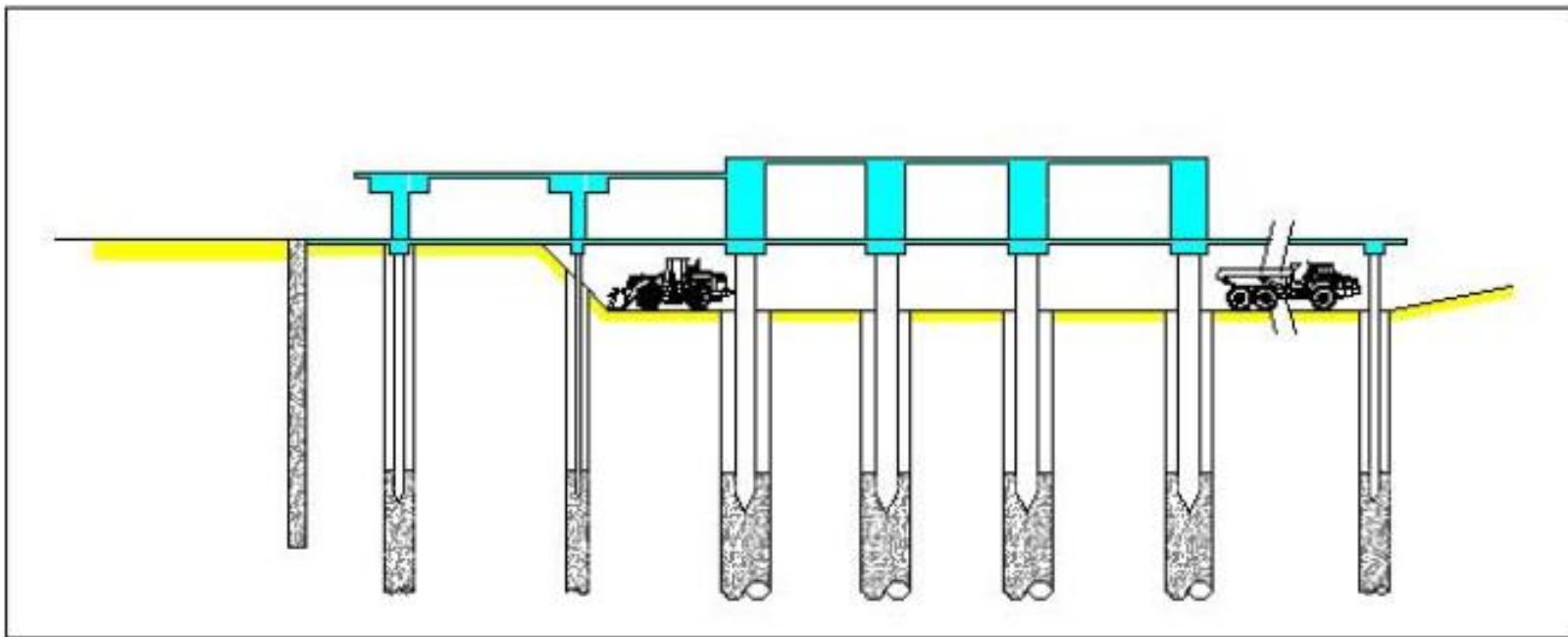
Some are only temporary and some are incorporated into the permanent works

Wynn's Resort, Macao



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Construction Sequence



- 1. Complete Construction of Lower Ground Floor Slab
- 2. Start Excavating below Slab to U/S of Basement B1
- 3. Start Construction of Ground Floor Slab

Voids in B Floors for access



Plunged columns

Cast into in situ

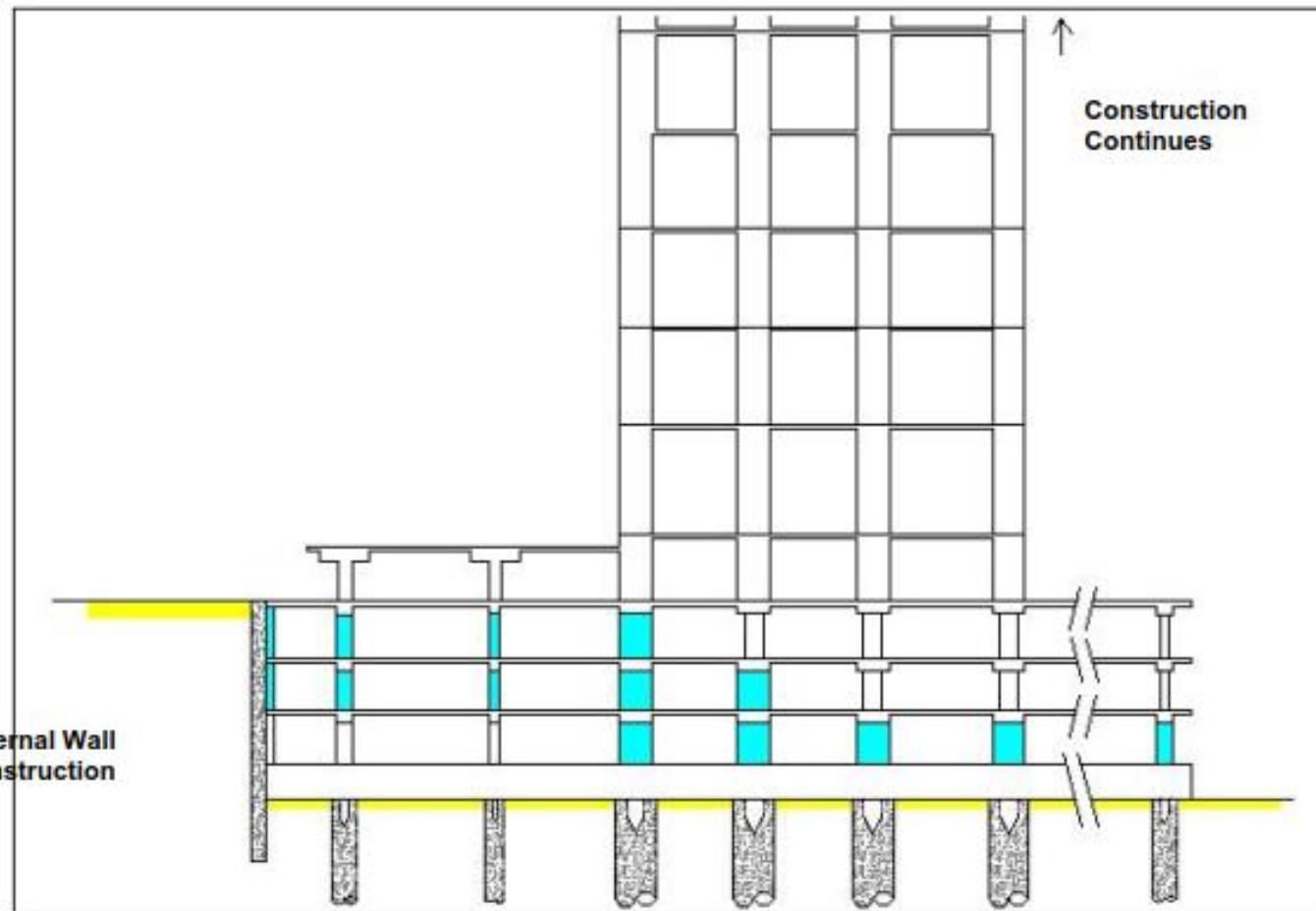


Lots of detailed design required



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Building limited to 8 floors until plunged columns are cased when B5 is reached



- 1. Complete Construction of External Walls and Internal Columns
- 2. Bottom-Up Construction of Superstructure Continues

Case 4: Partial Top-Down...example

Multiple roadways and access for hoisting



Case 5: Hong Kong Zuhai Macau Bridge

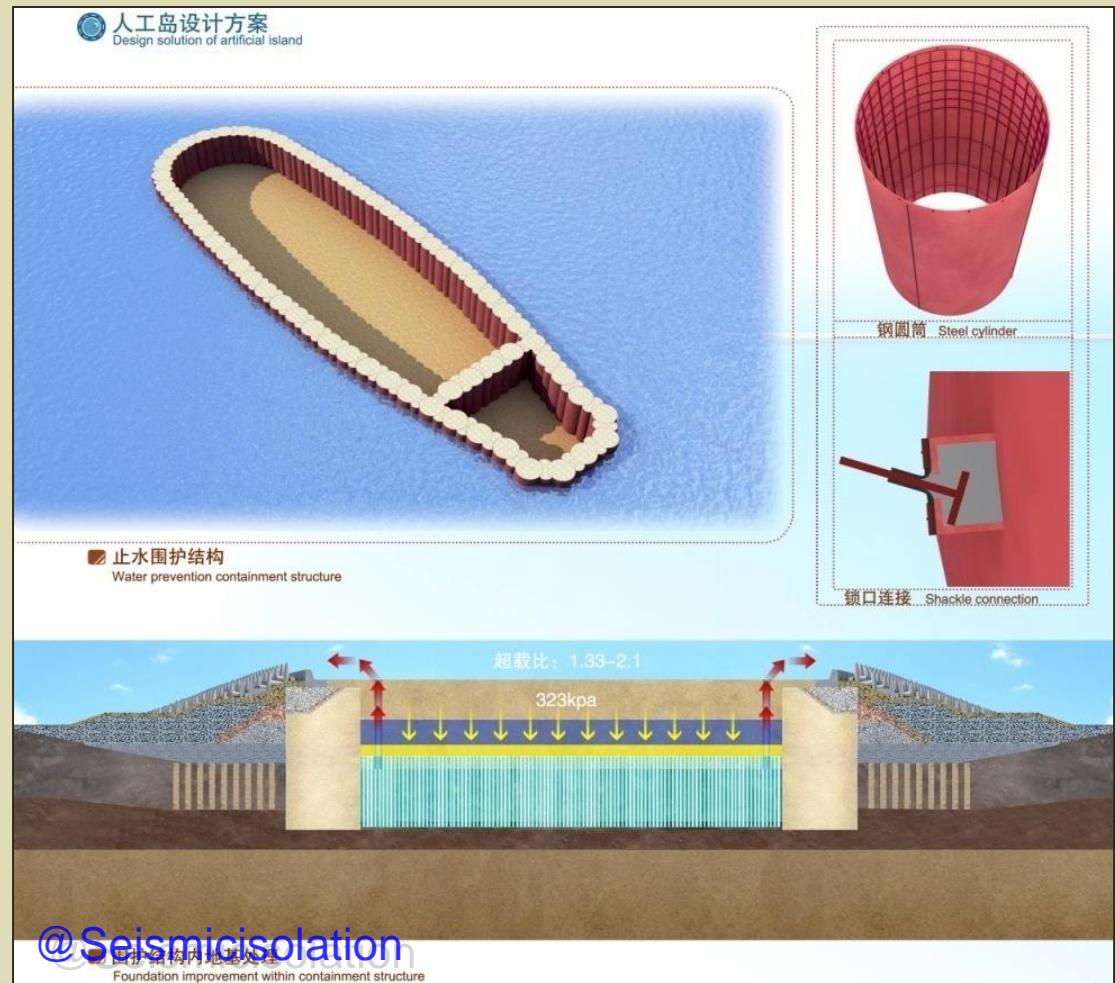
Man-made islands are portals for immersed tube tunnel

The Two man-made islands
of about **100,000 m²**.
(1 million square feet)

Water depth is about **10m**.

Thickness of soft soil layer is
20 ~ 30m.

Retaining structure is a cellular
cofferdam **22m diameter**,
average depth **45m** and filled
with sand.



Fabrication of Steel Cofferdam



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Assembly of Steel Tubes



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Combining Steel Tubes



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Transporting



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Installing by self-weight and six vibrators



Filling



Closing the retaining walls



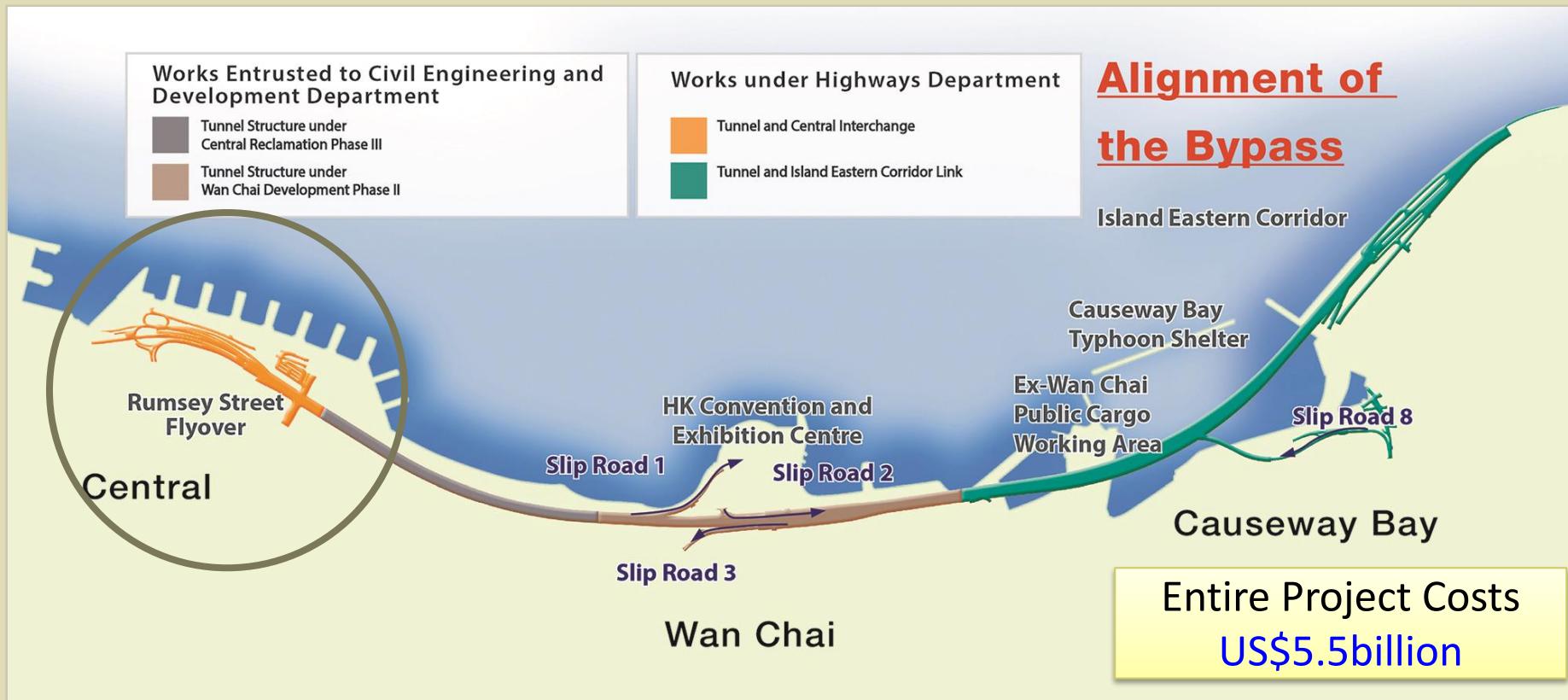
Filling the Island



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Case 6: Central - Wan Chai Bypass and Island Eastern Corridor Link

3km dual lane dual carriageway in cut and cover tunnels



Central District Portal



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Central - Wan Chai Bypass and Island Eastern Corridor Link



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Central - Wan Chai Bypass Middle Section



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Crossing yacht basin with man-made islands



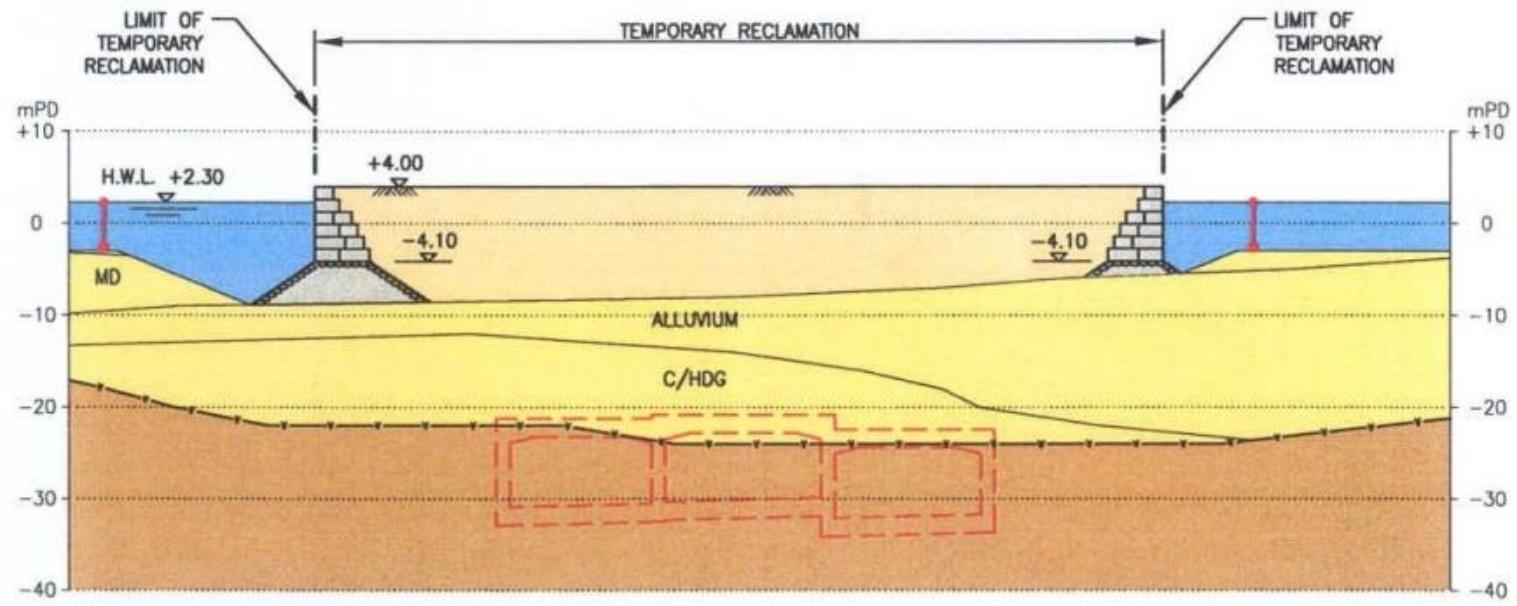
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Temporary sea walls



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Construction of man-made island



Temporary block sea walls and sand fill island

Excavation in the dry



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Complexity

Tunneling below existing road tunnel and new railway tunnel



Mining in rock



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Case 7:

Re-use of 35 years old “Temporary Walls”

1975 Requirements

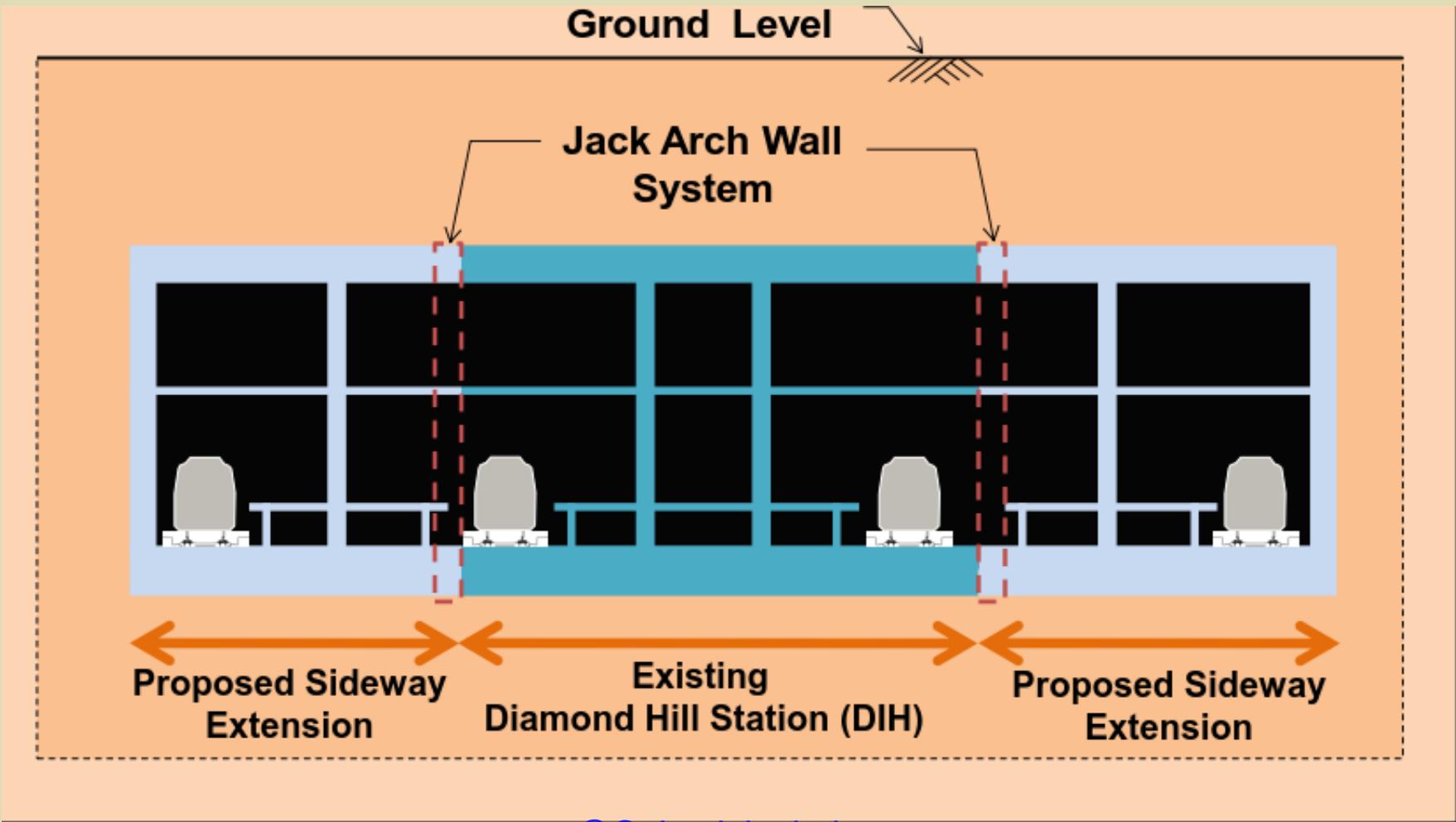
Short Term

- Central platform, two lines (standard) layout underground station.

Future

- Provision for future extension on both sides to provide a junction station with cross platform connections.
- Long side walls shall be easy to dismantle under normal operation of the lines (3 hours night time working)

1975 Intended future expansions



Contractor's design for temporary walls for uncertain duration

Jack Arch Wall with King Posts

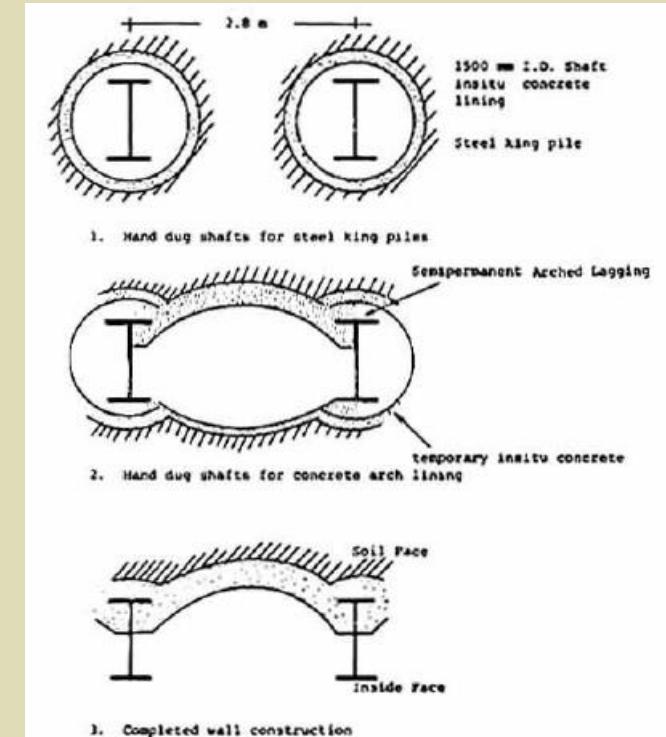
1 Hand dug shafts for plunged rolled steel sections (U.B. 911 x 418 mm).

2 Hand dug shafts for concrete lagging between steel U.B.s

3 plain concrete arches

Top down construction with dewatering as the shafts are sunk reducing lateral earth pressure

Plan view, stages of construction



Hand excavated shaft

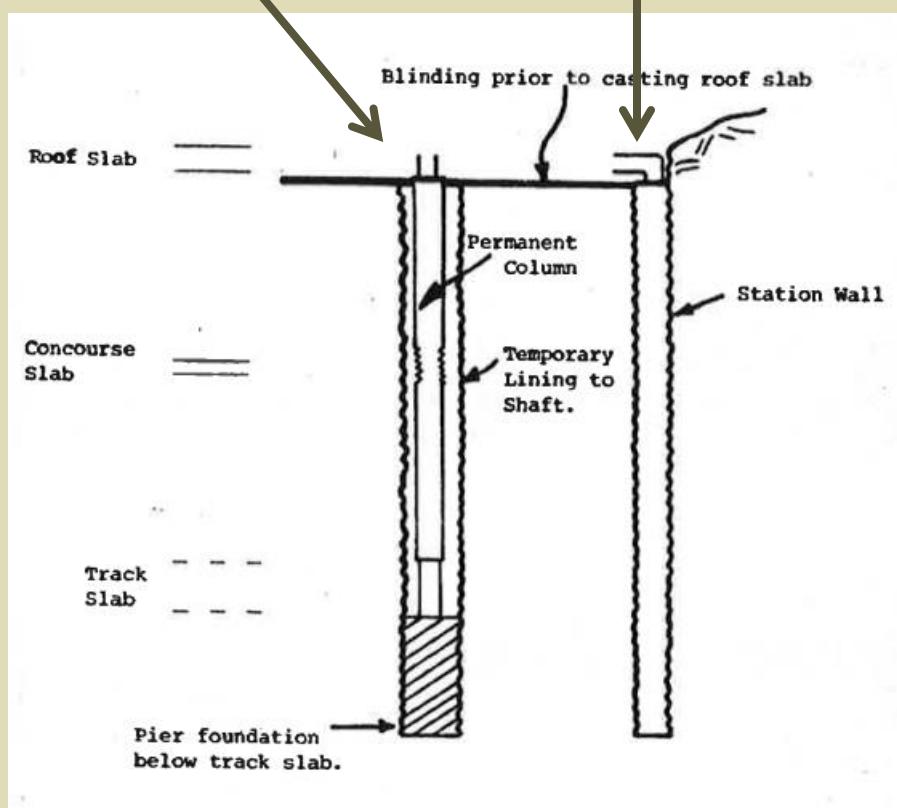


Discarded options

- Diaphragm walls would require a long time to break out whereas steel can be cut readily
- Heavy steel sheet piles would require driving past 3m dia. Boulders
- Sheet piles are insufficient in bending capacity over large spans.

Hand excavated shafts for plunge columns

Hand excavated shafts for walls



200 shafts excavated concurrently
Perimeter walls constructed in 80 days



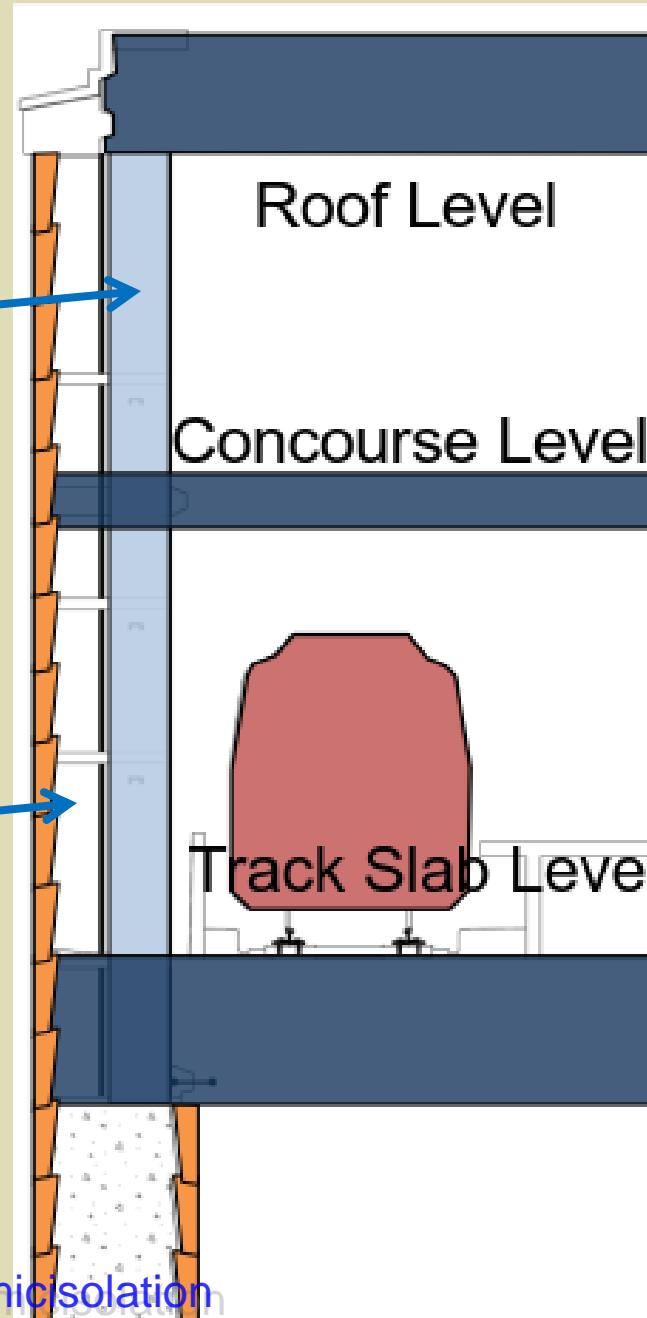
North Wall

Plunged Columns

Adopted design

Steel Beams
plunged into
hand dug shafts

Plain concrete
arches



Steel beams and concrete jack arches temporary walls



Contiguous hand ~~excavated~~ permanent concrete walls

1975 Calculations for soil-structure interaction by hand

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Earth Pressure and Stability of Slopes

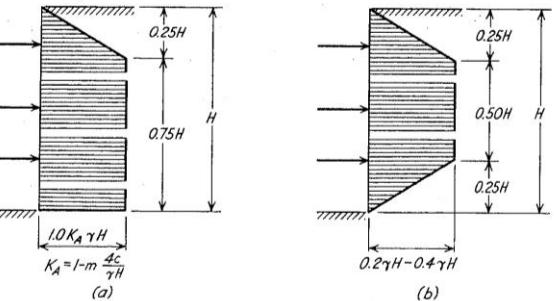
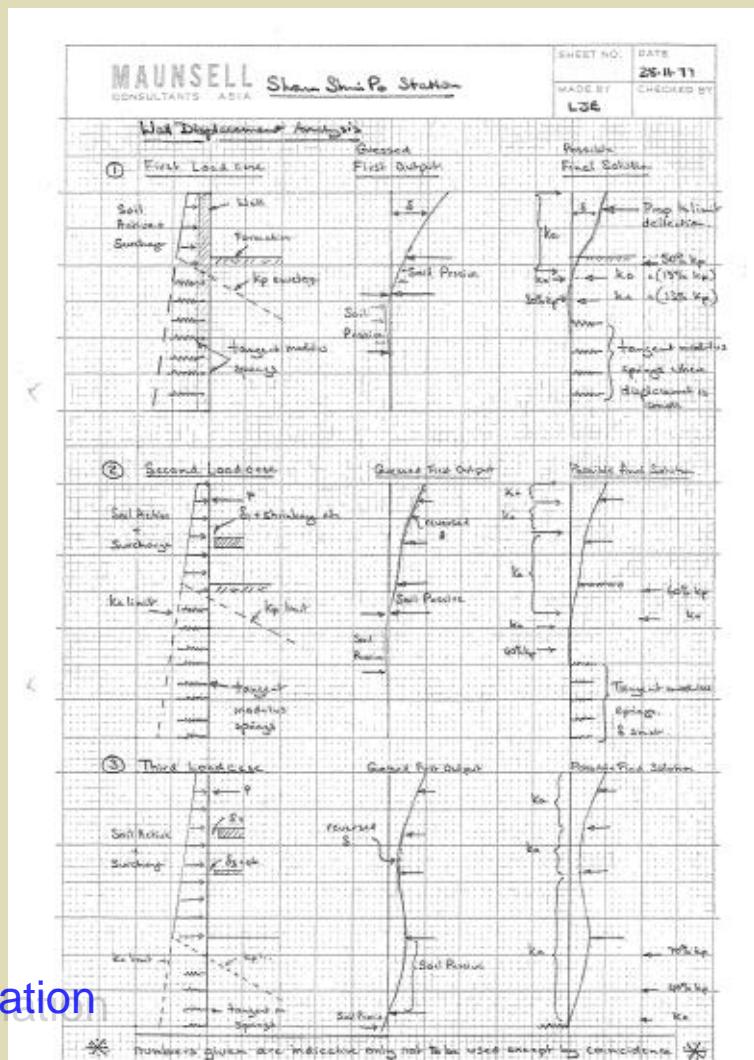
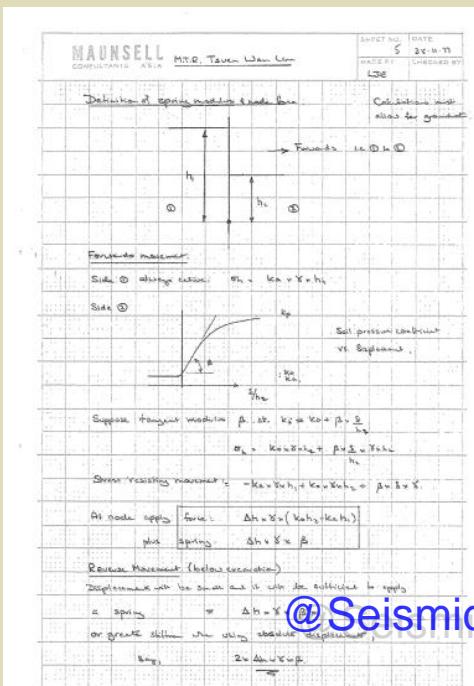


Fig. 48.11. Apparent pressure diagrams for design of struts in cuts excavated in clay soils. (a) Diagram for soft to medium clays; value of m to be taken as 1.0 except for truly normally loaded clays when $N = \gamma H/c$ exceeds about 4, under which conditions $m < 1.0$. (b) Tentative diagram for stiff-fissured clays; lower pressure to be used only when movement can be kept to a minimum and construction time is short.



1975 Plastic design for steel

Welding
RESEARCH

The Full Plastic Moments of Sections Subjected to Shear Force and Axial Load

By M. R. Horne, Sc.D., Ph.D., A.M.I.C.E.

SYNOPSIS

Existing methods of calculating the full plastic moments of sections under axial load only, and under shear force only are reviewed. An approximate but safe formula is presented for full plastic moments under combined axial loads and shear forces. Consideration is given to the limiting depth to thickness ratios which may be allowed in the web. A comparison is made between solutions for the elastic buckling of webs under various conditions with the limiting depth to thickness ratios derived at experimentally as suitable for members forming part of a structure designed by the plastic theory. A design formula is suggested that gives the maximum permissible depth to thickness ratio in terms of the mean yield stress in the web.

Introduction

The effect of axial loads on the bending moments required to produce full plasticity is well understood. The expression for the full plastic moments of members of rectangular section was given by Baker,¹ and a similar treatment is readily extended to members of I-section.² Tables giving direct expressions for the full plastic moments of standard rolled sections under axial load are available.³ The general case of unsymmetrical sections subjected to bending moments and axial loads has been discussed fully by Eickhoff.⁴

The influence of shear forces on the moments at full plasticity has been discussed by Horne,⁵ Green,⁶ and Heyman and Dutton.⁷ Horne offers a lower bound (conservative) solution, based on the Tresca yield criterion (maximum stress difference), according to which any shear force reduces the full plastic moment. Green gives an upper-bound (unconservative) solution based on the von Mises criterion (strain energy due to shear). He shows that the effect of shear forces depends on the method of support at the 'plastic hinge' sections, and obtains two solutions, for 'strong' and 'weak' supports respectively (Fig. 1). Green maintains that, due to the restraining effect of the support material, the full plastic moment remains above the value given by the

simple theory ignoring shear, unless the mean shear stress is greater than about one-third the yield stress in pure shear for a weak support, or half this yield stress for a strong support. Although these upper bound solutions may be unsafe, since they are not accompanied by stress distributions for the uniaxial material, Green considers them to be sufficiently accurate for practical purposes.

A design formula for calculating the effect of shear forces on the full plastic moments of I-sections has been given by Heyman and Dutton.⁷ The formula is obtained from an approximate solution based on the von Mises yield criterion. It is slightly more conservative than the solution given by Horne,⁵ but applicable over a larger range of shear force. It does not allow for the beneficial effect of restraint at the hinge positions noted by Green.⁶ While it would be most satisfactory if this restraint could be taken into account, it is probably unsafe to do so in most structural problems. The 'weak support' case considered

Report FE 1/51/57 of the B.W.R.A. issued in March 1957.
The author is with the Department of Engineering, University of Cambridge.

APRIL 1976

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Fig. 2

HORNE'S FULL PLASTIC MOMENTS OF SECTIONS

Green is not necessarily the worst condition which might arise, and it is therefore preferable to accept a safe solution. Heyman and Dutton's proposal, besides being conveniently simple, has a direct physical interpretation, and may therefore be accepted as the most suitable tools for design. Although effects are not additive in plasticity as in elasticity, the influence of axial load and shear force is, if considered, best dealt with separately. A suitable procedure, based on extensive tests, has been recommended by Rederick and Phillips,⁸ and involves the requirement that full plasticity shall be reached at a distance away from a concentrated load or support equal to half the depth of the member.

The effect of axial loads when combined with shear forces on the bending moments required to cause full plasticity has been discussed by Green⁶ in relation to the plane strain problem, but not in relation to plane stress. This latter condition is the one applicable to the usual types of structural member and so Green's solutions are not relevant. The purpose of this report is to present a design formula which will give the full plastic moment of a structural member in the presence of an axial load together with a shear force acting parallel to the web. The question of the maximum depth to thickness ratios permissible in webs subjected to combined shear force, axial load, and bending moment is also discussed.

BENDING MOMENT IN THE ABSENCE OF AXIAL OR SHEAR FORCES

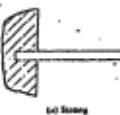
The stress distribution at full plasticity in the web of an I-section member subjected to a pure bending moment about the major axis XX' (Fig. 2) is shown in Fig. 3 where (a) represents longitudinal stresses and (b) shear stresses (zero in this case) acting on a vertical plane. If the yield stress in tension and compression is f_y , the moment of resistance of the web, M_w , is

$$M_w = f_y \frac{\pi}{4} d^2 \quad (1)$$

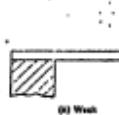
where t is the thickness and $2d$ the depth of the web (Fig. 2).

BENDING MOMENT IN THE PRESENCE OF AN AXIAL LOAD

When an axial load P acts in conjunction with a bending moment M_w , the stress distribution at full plasticity is as shown in Fig. 4. If f_u is the mean axial stress calculated on the area of the web only (i.e. $f_u = P/2td$) and y is the distance of the neutral axis from the central axis XX', it is readily shown that y and M_w are given respectively by



(a) Strong



(b) Weak

Fig. 1—Strong and weak support conditions as classified by Green

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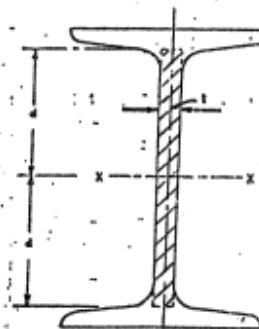


Fig. 2—Web dimensions in an I-section member

$$M_w = \left(f_y - \frac{f_u}{2} \right) dt \quad (2)$$

BENDING MOMENT IN THE PRESENCE OF A SHEAR FORCE

The stress distribution assumed by Heyman and Dutton⁷ when full plasticity is caused by a combination of moment and shear force is shown in Fig. 5. The total shear force (denoted by G) is assumed to be uniformly distributed down the web (Fig. 5b), reducing a stress $\sigma_w = G/2td$. The longitudinal tensile and compressive stresses are reduced from f_y to f_u , where

$$f_u = f_y + 2\sigma_w \quad (3)$$

The moment of resistance M_w becomes

$$M_w = f_u \frac{\pi}{4} d^2 \quad (4)$$

The relationship between f_u and σ_w is shown graphically in Fig. 7 for a yield stress f_y of 15.25 tons/in.²

BENDING MOMENT IN THE PRESENCE OF AN AXIAL LOAD AND SHEAR FORCE

The stress distribution assumed when bending moment, axial load, and shear force all combine to produce full plasticity is shown in Fig. 6. The distributions in Figs. 3, 4, and 5 appear correctly as special cases of this general condition. In Fig. 6a, the longitudinal stress $\pm f_u$ is given by

$$f_u = f_y + 2\sigma_w + 2\sigma_s \quad (5)$$

while the distance y is given by

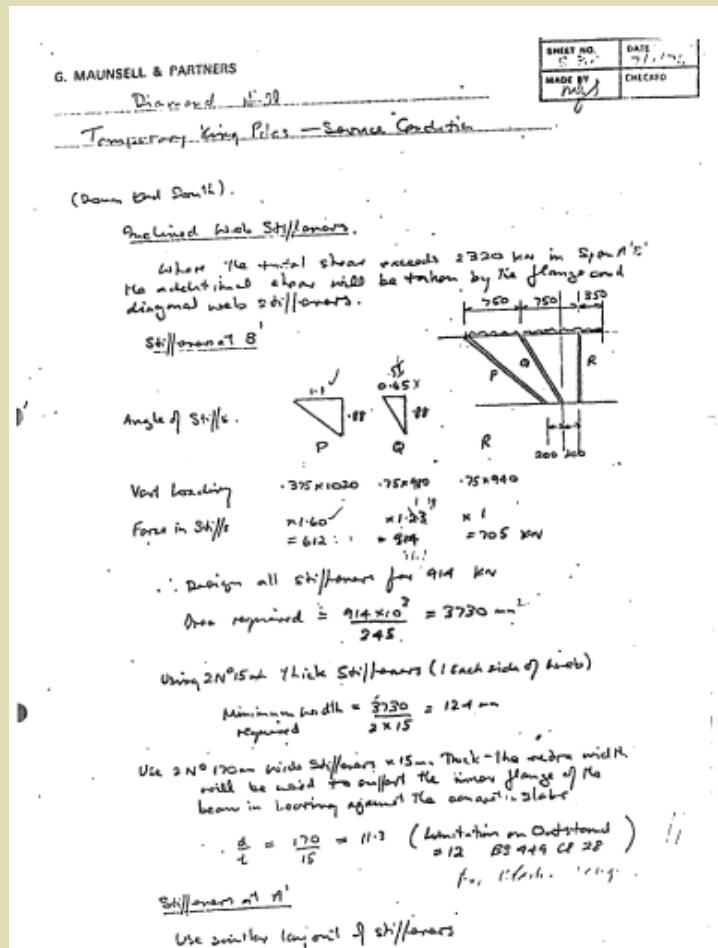
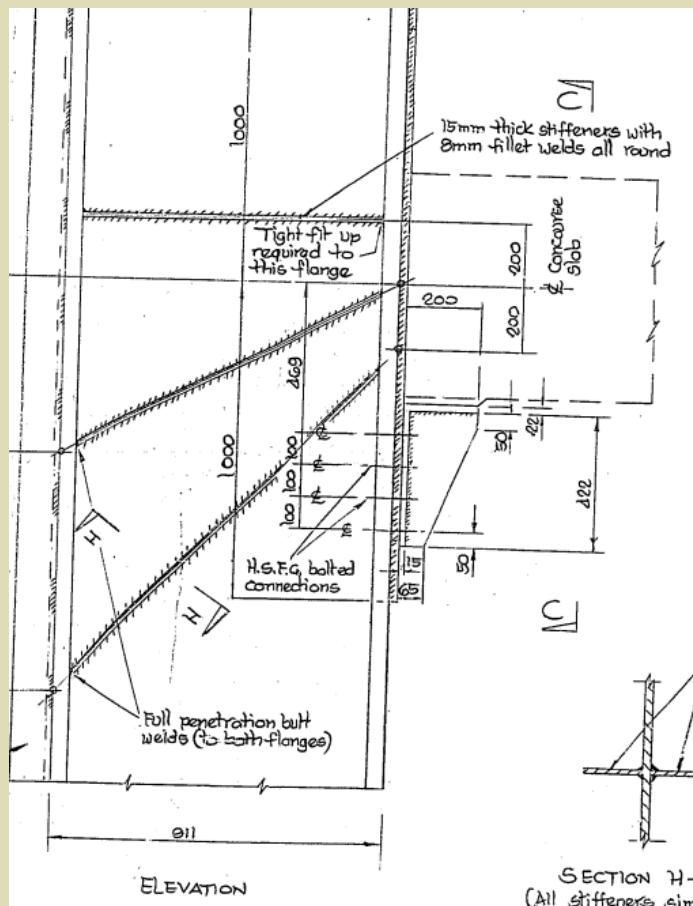
$$y = \frac{G}{2} \frac{d}{t} \quad (6)$$

The total moment of resistance M_w is

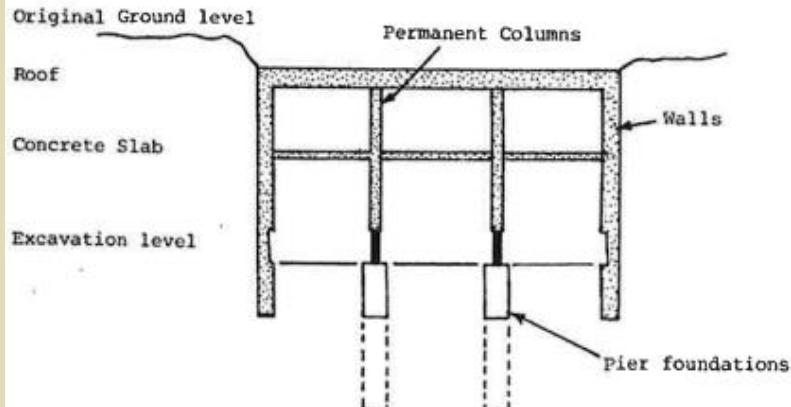
$$M_w = \left(f_y - \frac{f_u}{2} + \frac{G}{2} \right) dt \quad (7)$$

BRITISH WELDING JOURNAL

Steel detailing: Bending plus Shear



STATION SECTION AT FINAL EXCAVATION STAGE



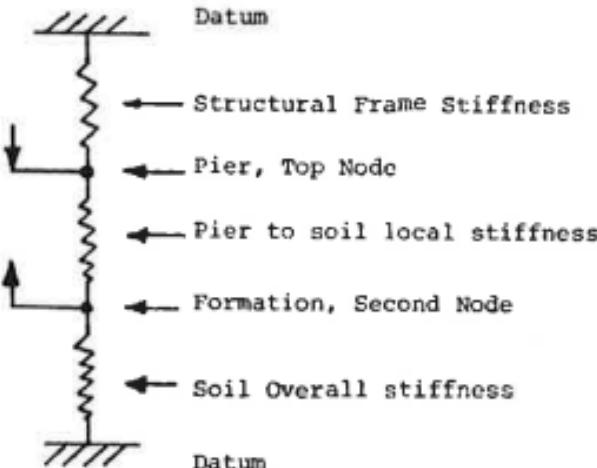
STRUCTURAL ANALOGUE

400 tons

2000 tons

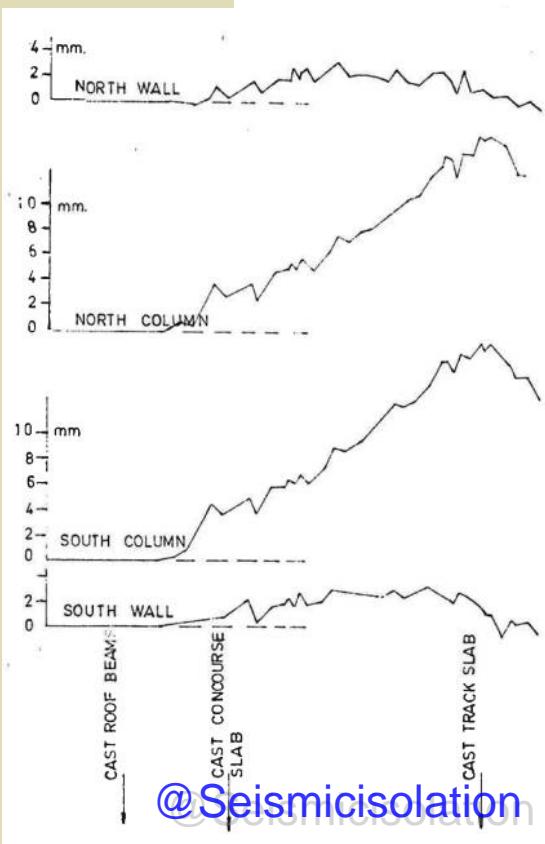
Structure load on Column

Relief due to excavation



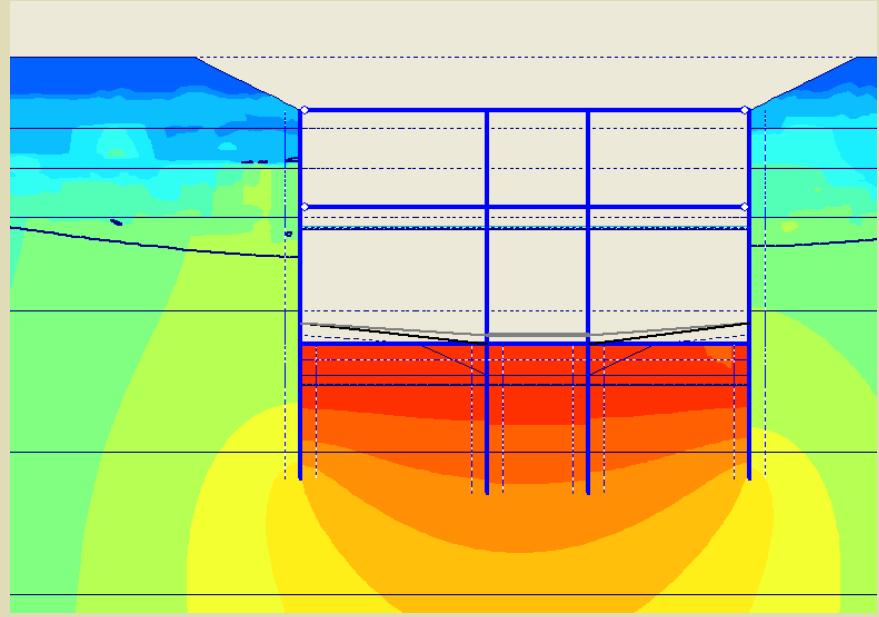
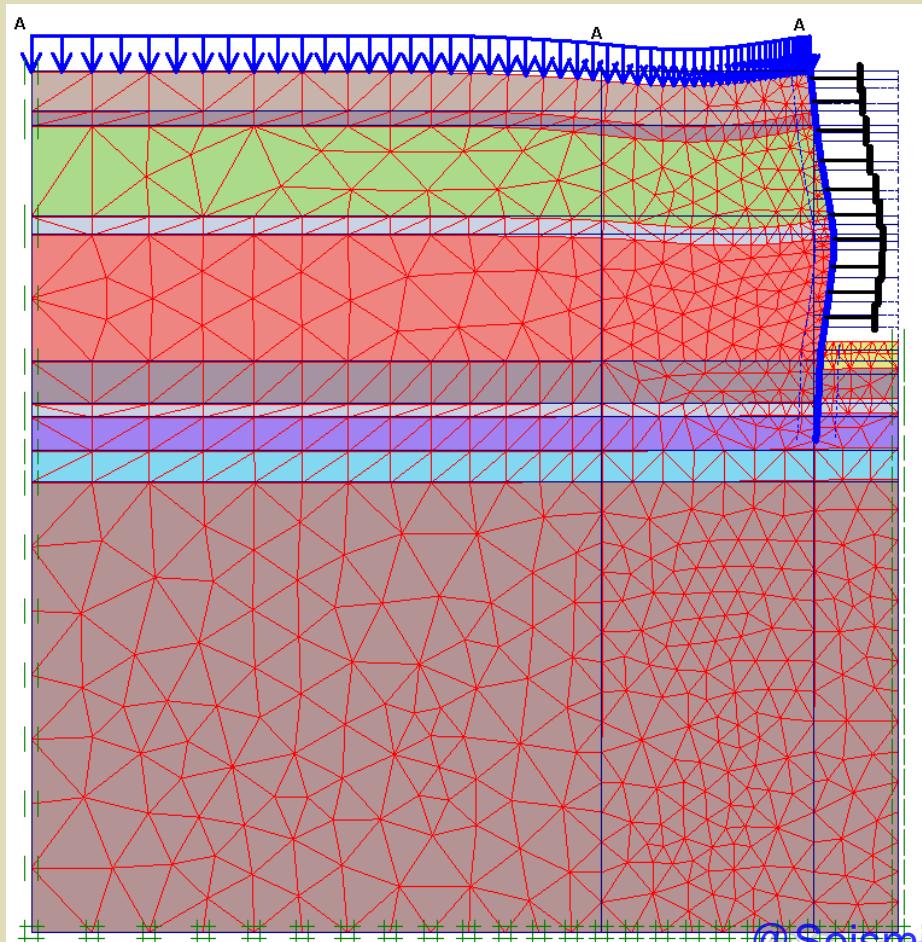
Vertical soil/structure interaction

Inadequate pier
foundations?

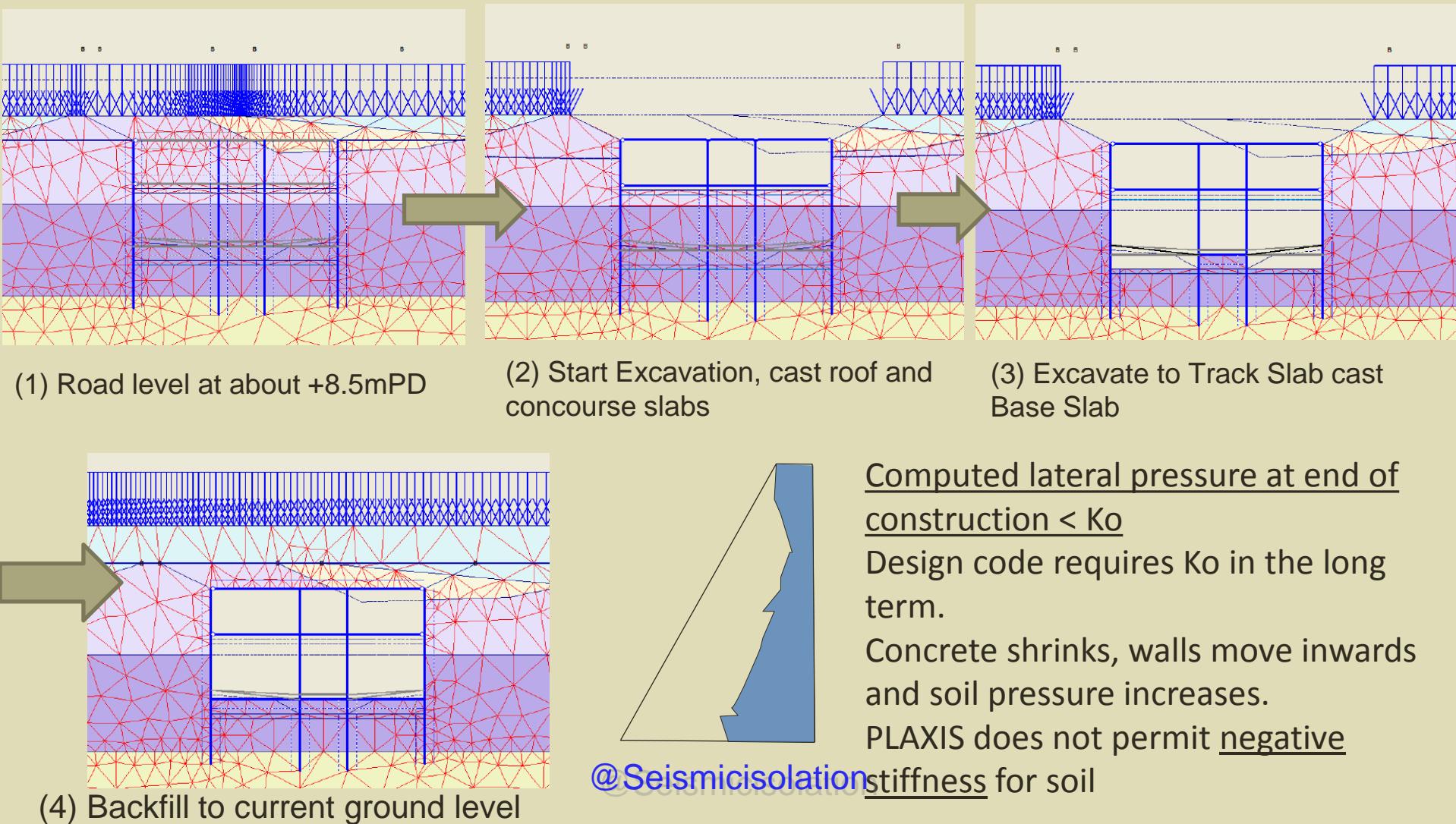


20 mm Heave

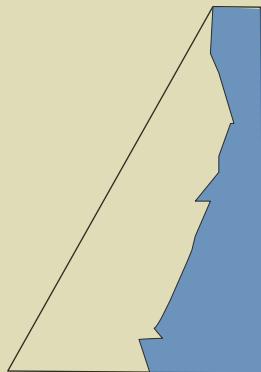
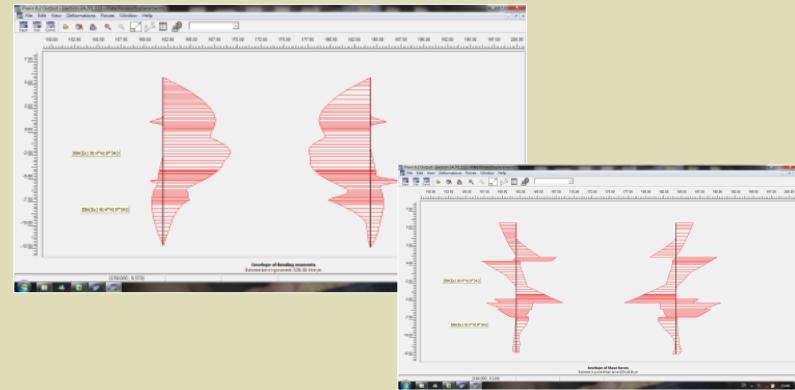
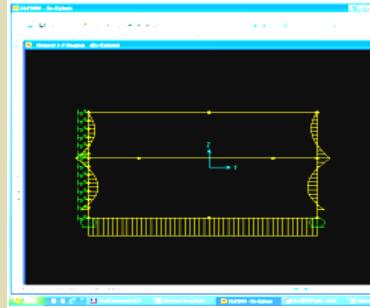
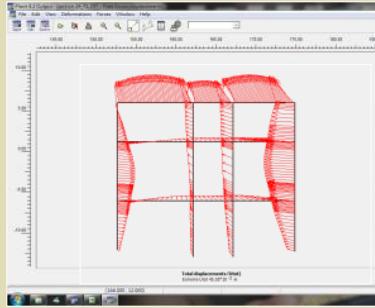
Re-evaluation: PLAXIS analysis of original construction



New analysis of original construction to estimate locked-in deformation after construction

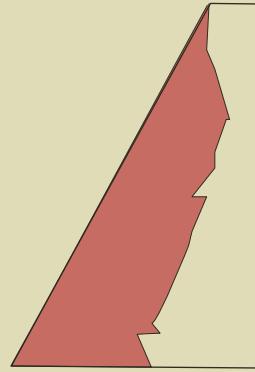


Permanent Case Structural Analysis - Present Superimposition of K_{PLAXIS} and $K_{O-PLAXIS}$



K_{PLAXIS}

+



$K_{O-PLAXIS}$

=



K_o

PLAXIS derived “Locked-in Stresses” due to Construction

SAP2000 to evaluate the Transition Stresses from transient active stage to permanent static stage (K_o -
Plaxis)

Loads are combined for checking long term case

@Seismicisolation

	Combined Stress	
	BS5950:1990	HK COP2005 Steel Code
Moment and Axial Force Utilization Factor		0.91 (Max)

Analysis and Condition Survey
confirmed
serviceability as permanent walls



Thank you