7. Seismic Design of Underground Structures

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Part A:
Earthquakes and Underground Structures

Oct. 2016

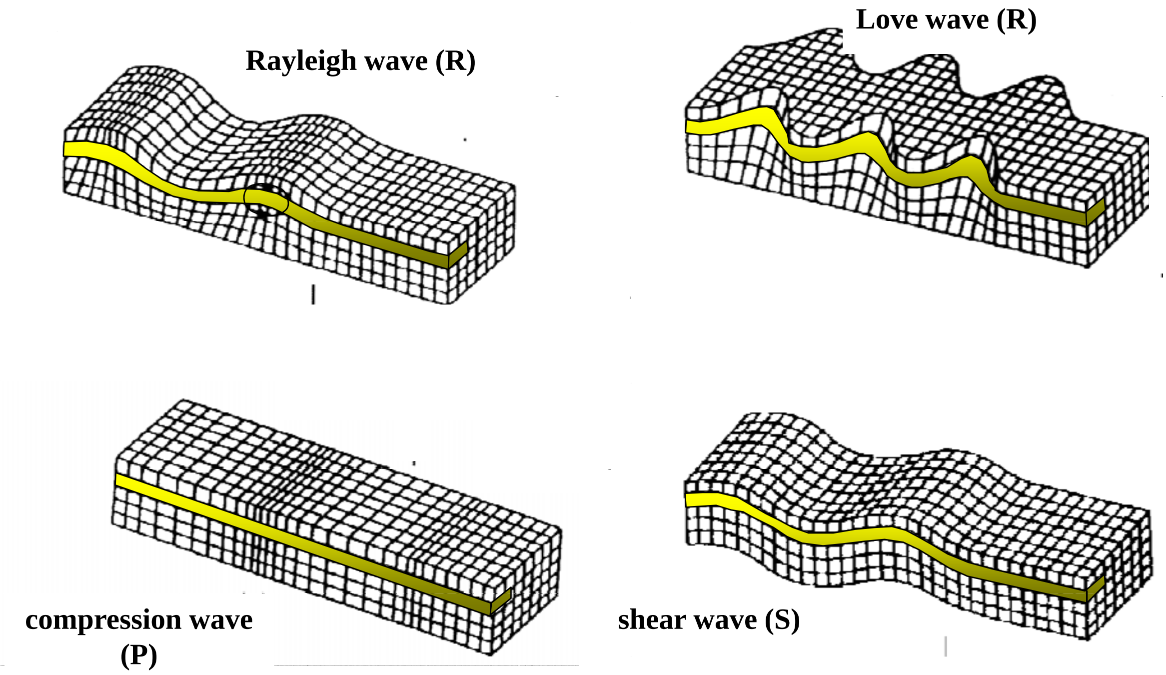
A. Earthquakes and Underground Structures

B. Seismic design against "transient" displacements

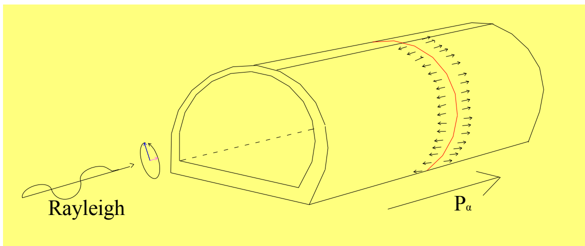
C. Seismic design against "permanent" displacements

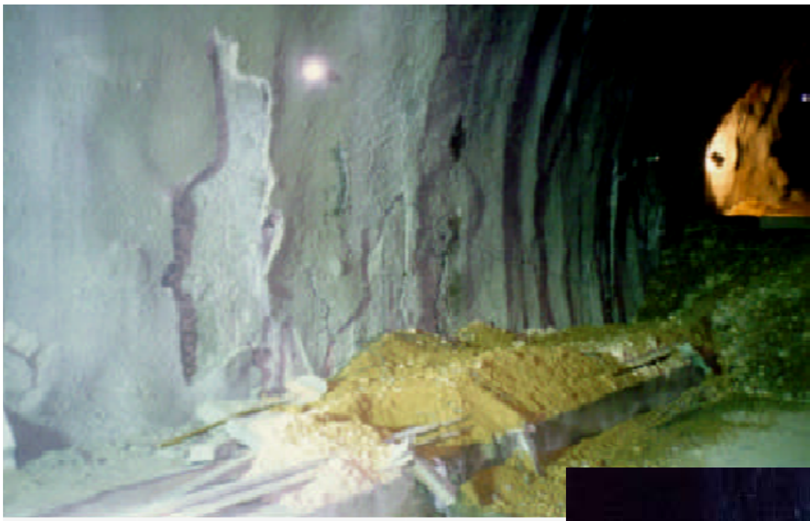
earthquake-induced transient ground displacements...

... are attributed to seismic wave propagation

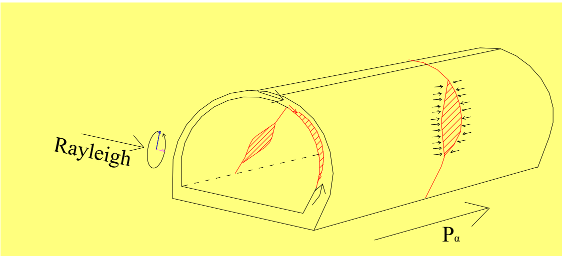


- peripheral cracks

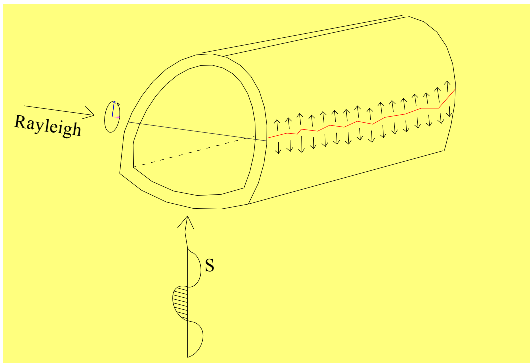


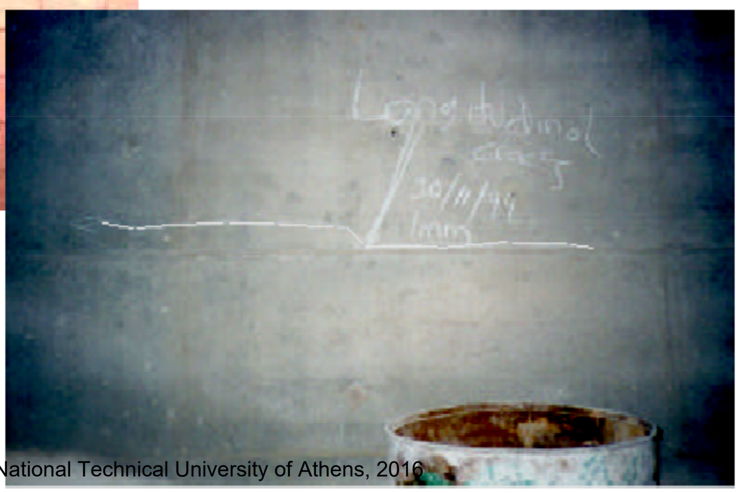


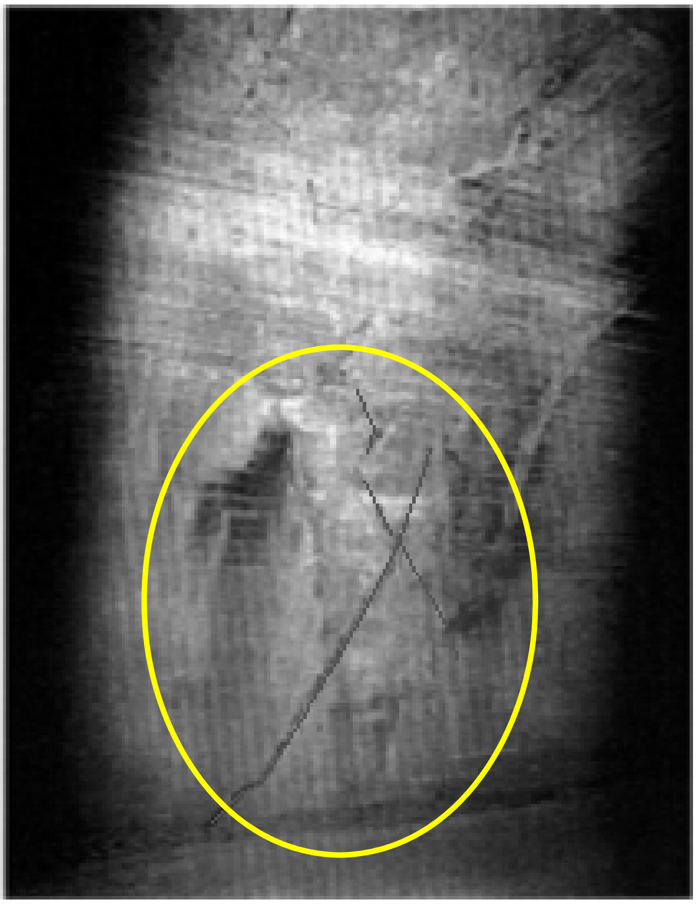
➢concrete spalling (compression)



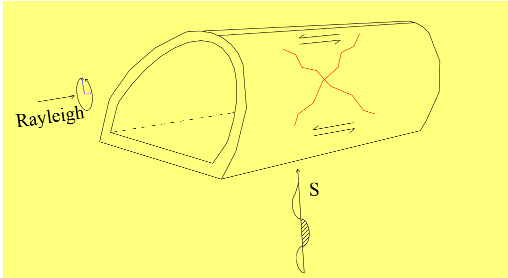
➢ longitudinal cracks



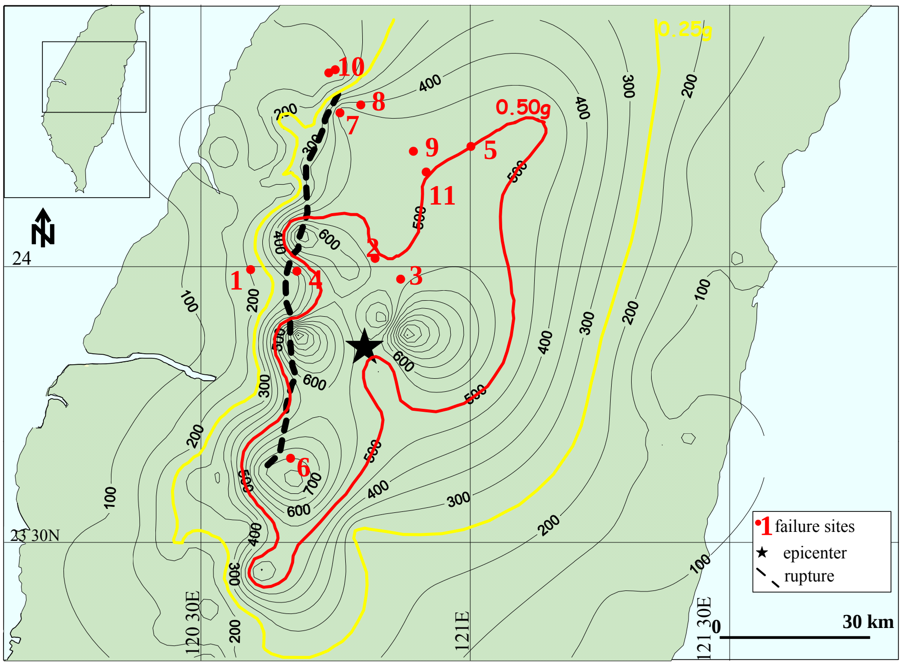




➢shear cracks

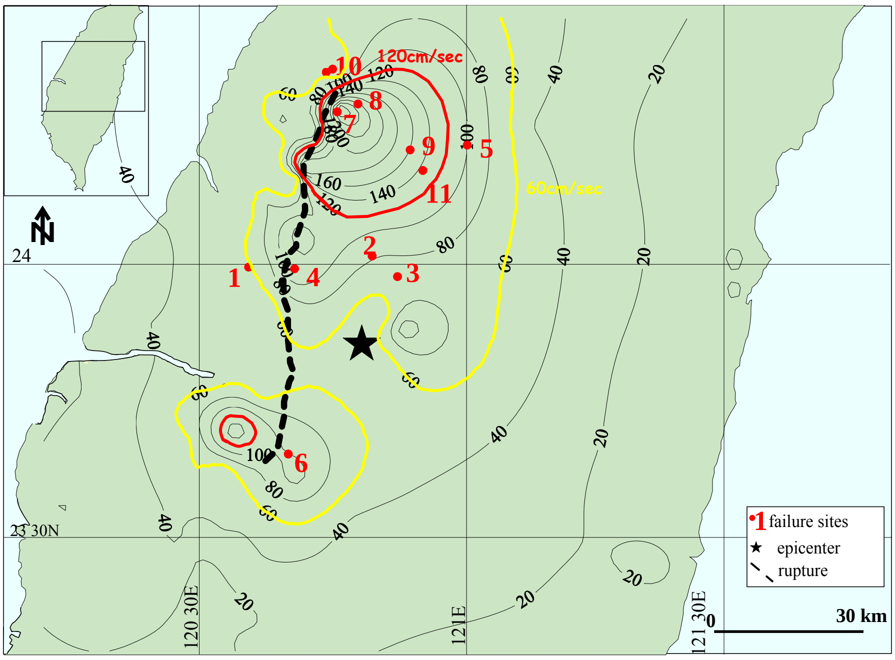


failure potential due to seismic wave propagation effects…



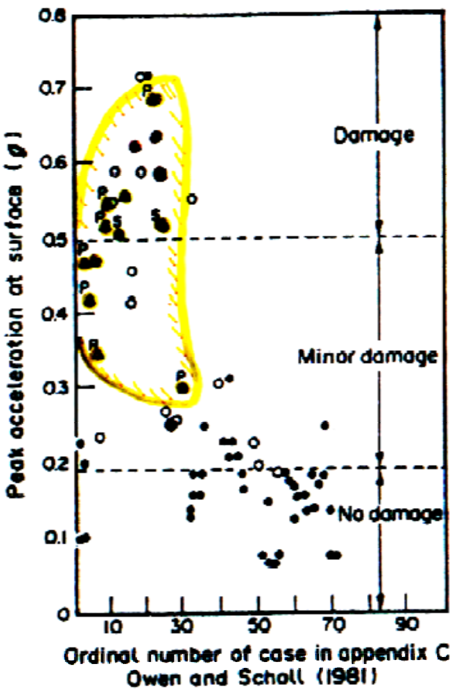
Locations of underground structures (concrete tunnels) that failed during the **CHI-CHI** (1999) earthquake, plotted against recorded maximum ground **acceleration** contours at ground surface.

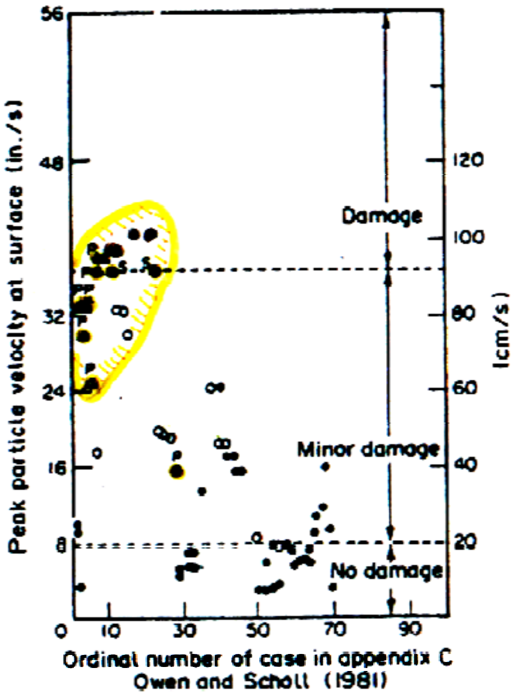
failure potential due to seismic wave propagation effects...



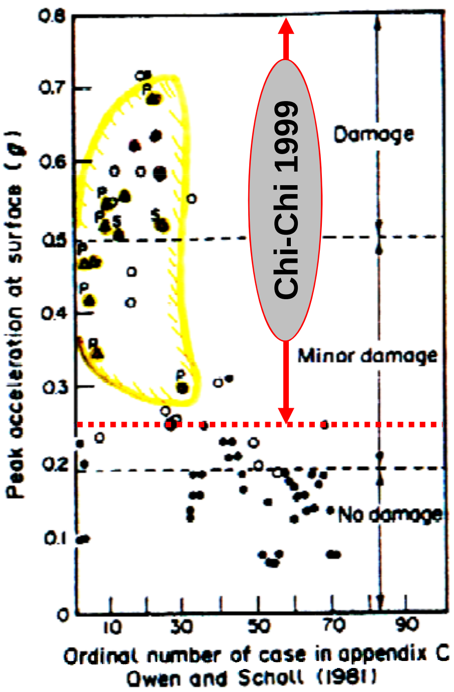
Locations of underground structures (concrete tunnels) that failed during the **CHI-CHI** (1999) earthquake, plotted against recorded maximum ground **velocity** contours at ground surface.

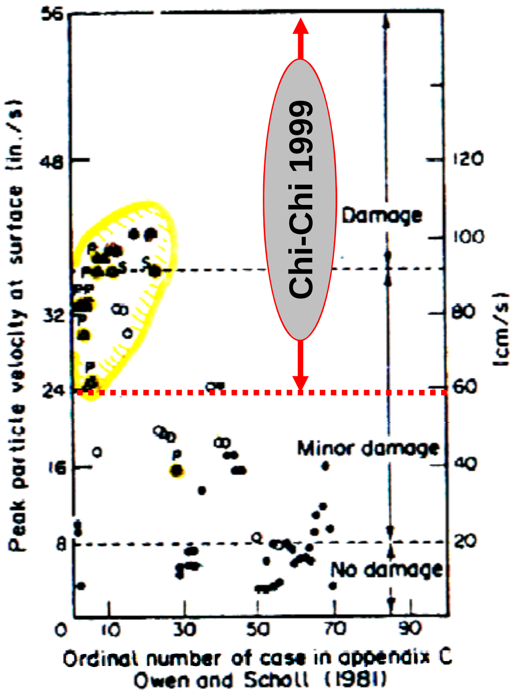
Correlation of strong motion levels with underground structure failures





Correlation of strong motion levels with underground structure failures

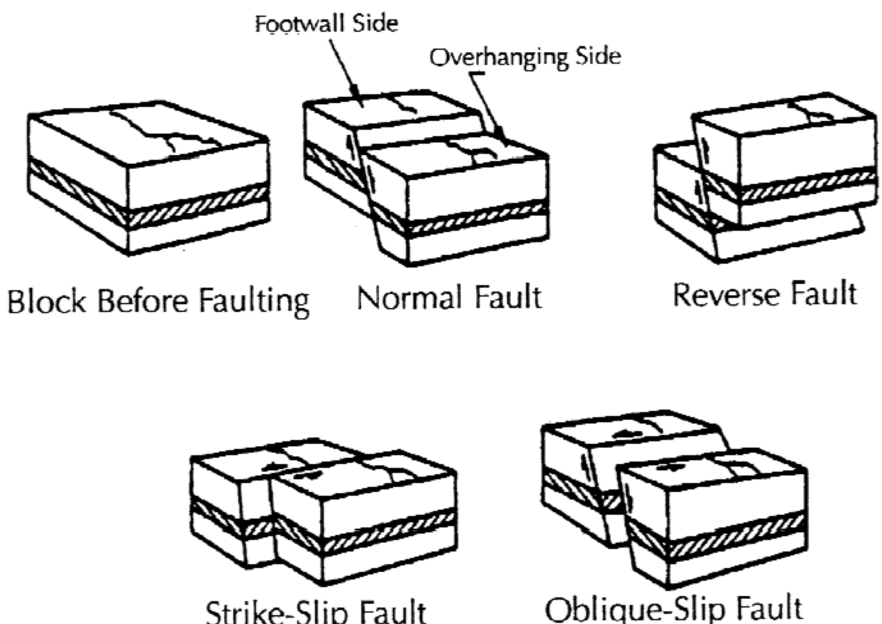




earthquake-induced permanent ground displacements...

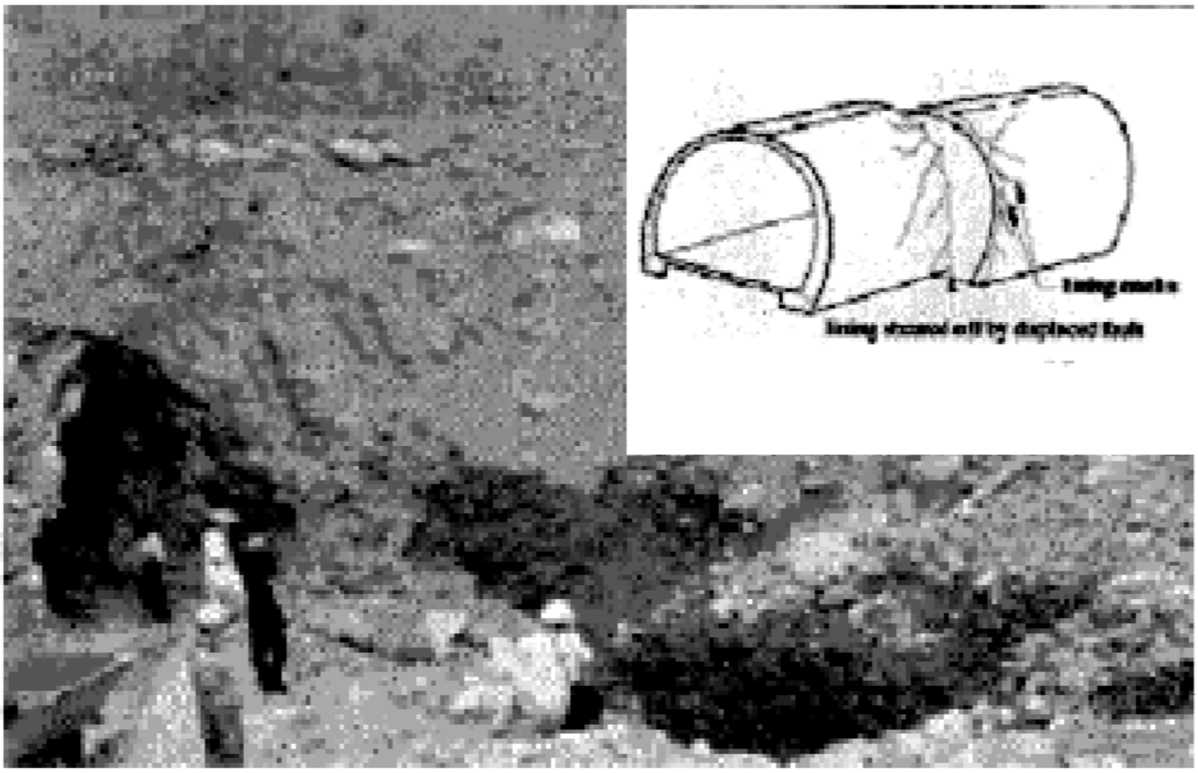
...are due to ground failure attributed to seismic effects. Typical examples include **fault rupture** propagation to the ground surface, **landslides**, **steep slope failures**, and mild slope failures due to liquefaction **(lateral spreading).**

fault rupture



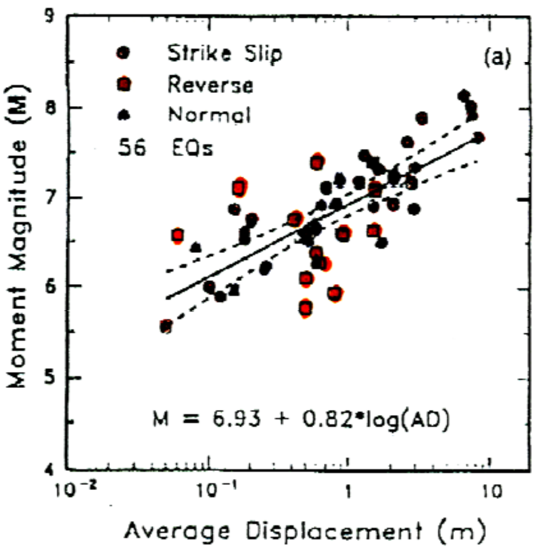
Tunnel collapse due to fault rupture propagation

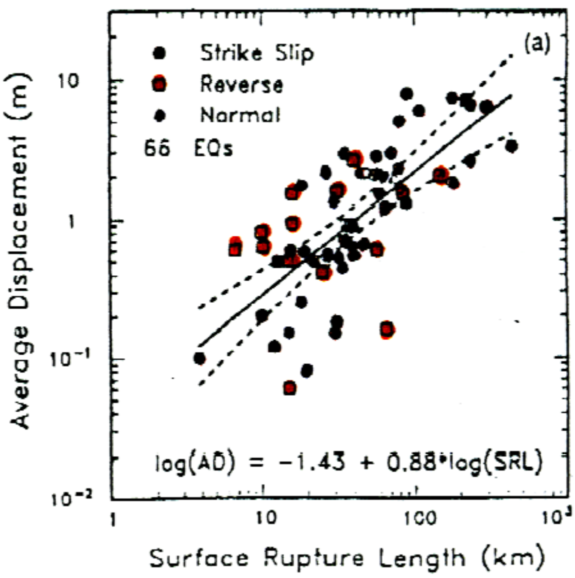
(Shih-Gang dam tunnel, Chi-Chi, 1999)



Empirical relations for the determination of the MEAN anticipated ground displacement due to fault rupture (Wells & Coppersmith, 1994).

*The **MAXIMUM** anticipated displacement is about twice the mean value*

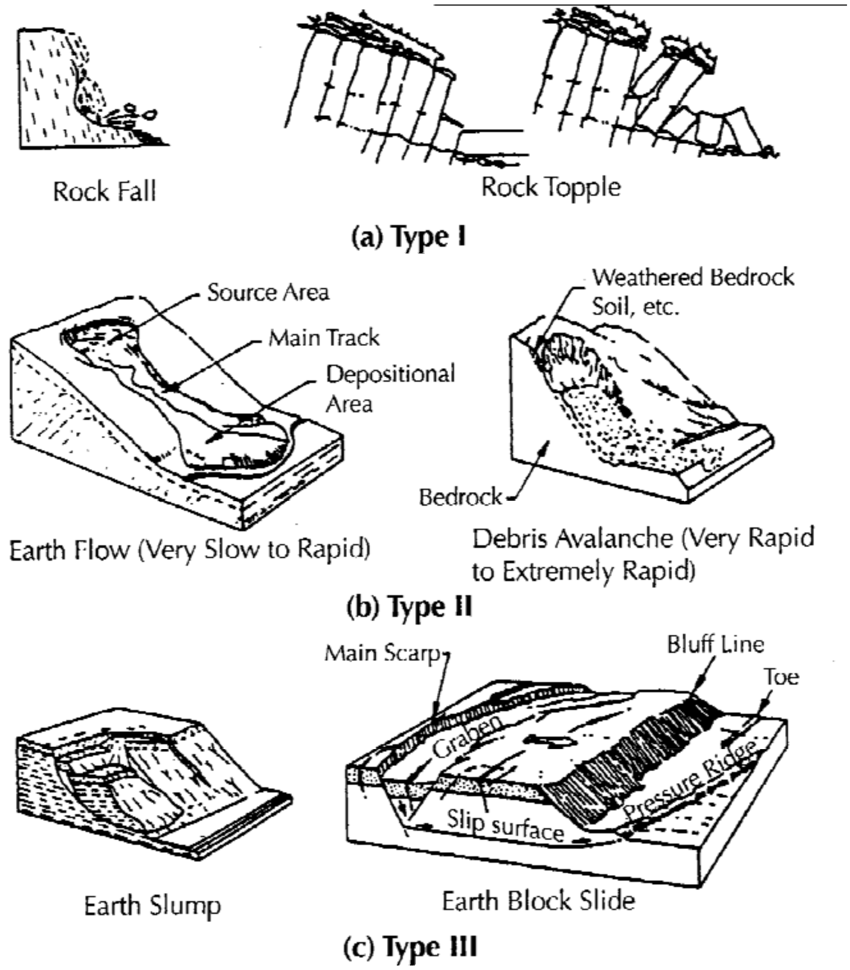


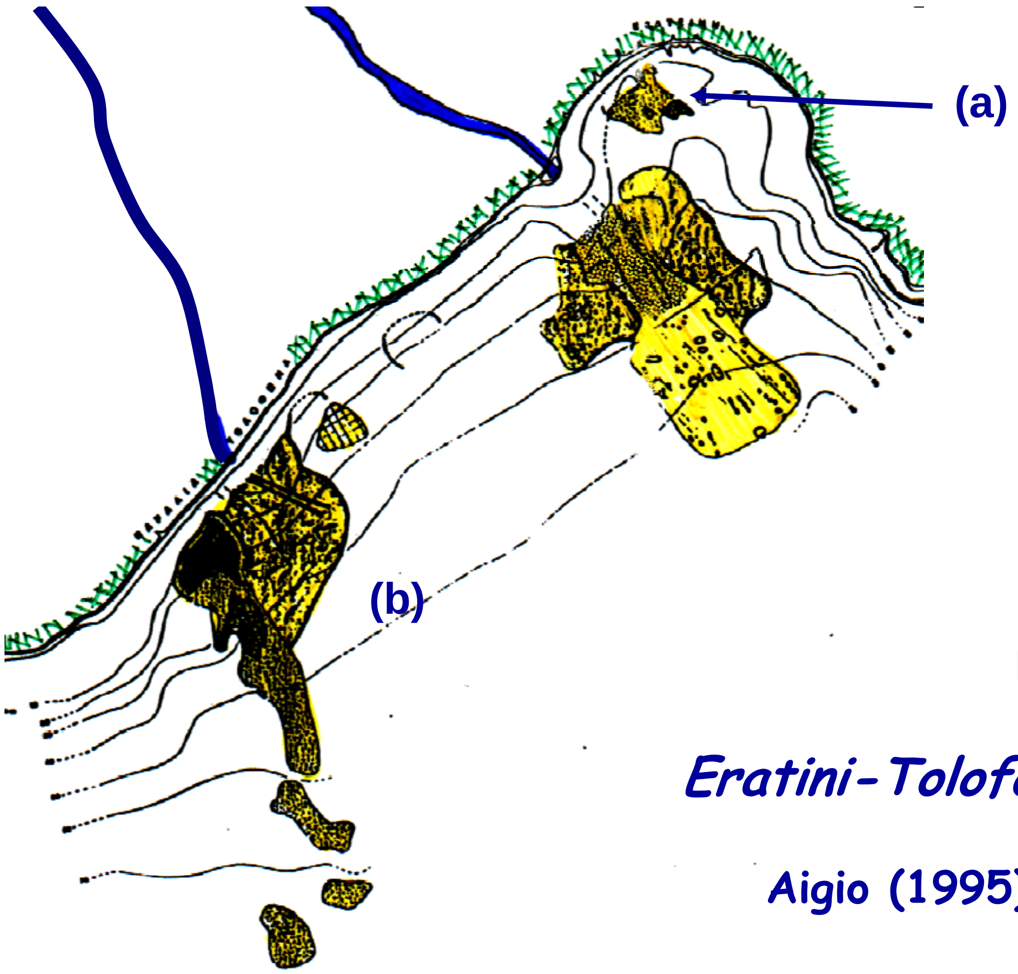


Permanent displacements

due to

SLOPE FAILURE

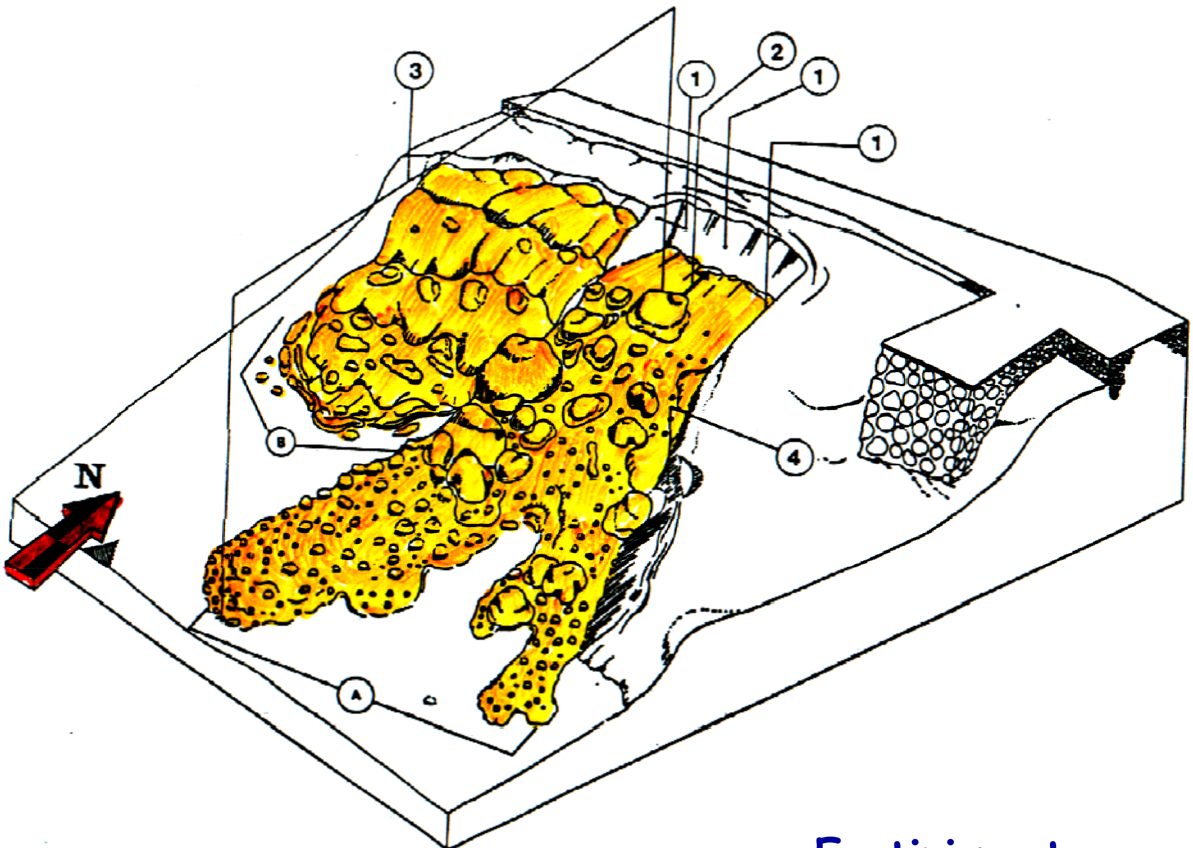




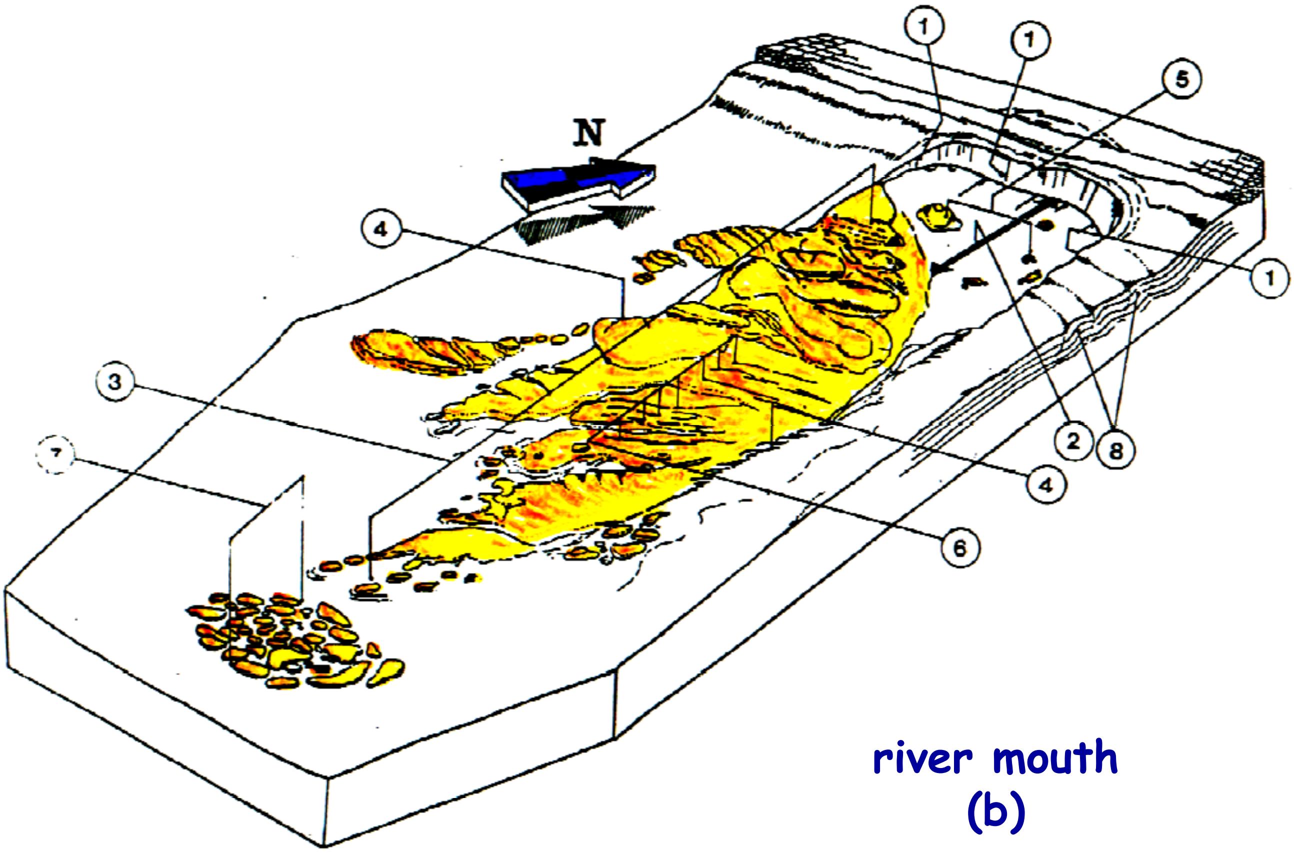
sub-sea
landslides at

Eratini-Tolofonas beach

Aigio (1995) earthquake



Eratini port
(a)

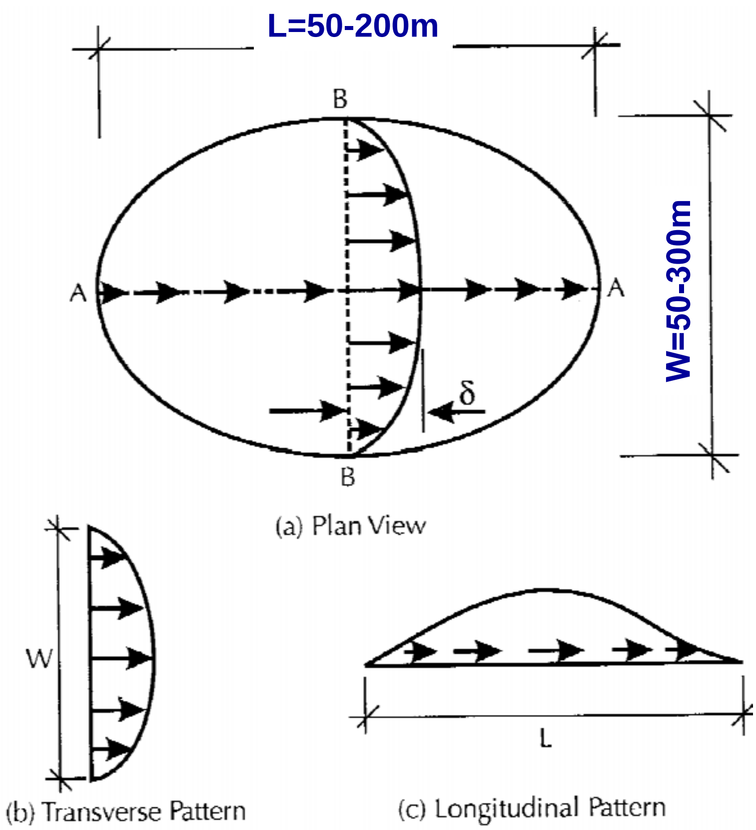


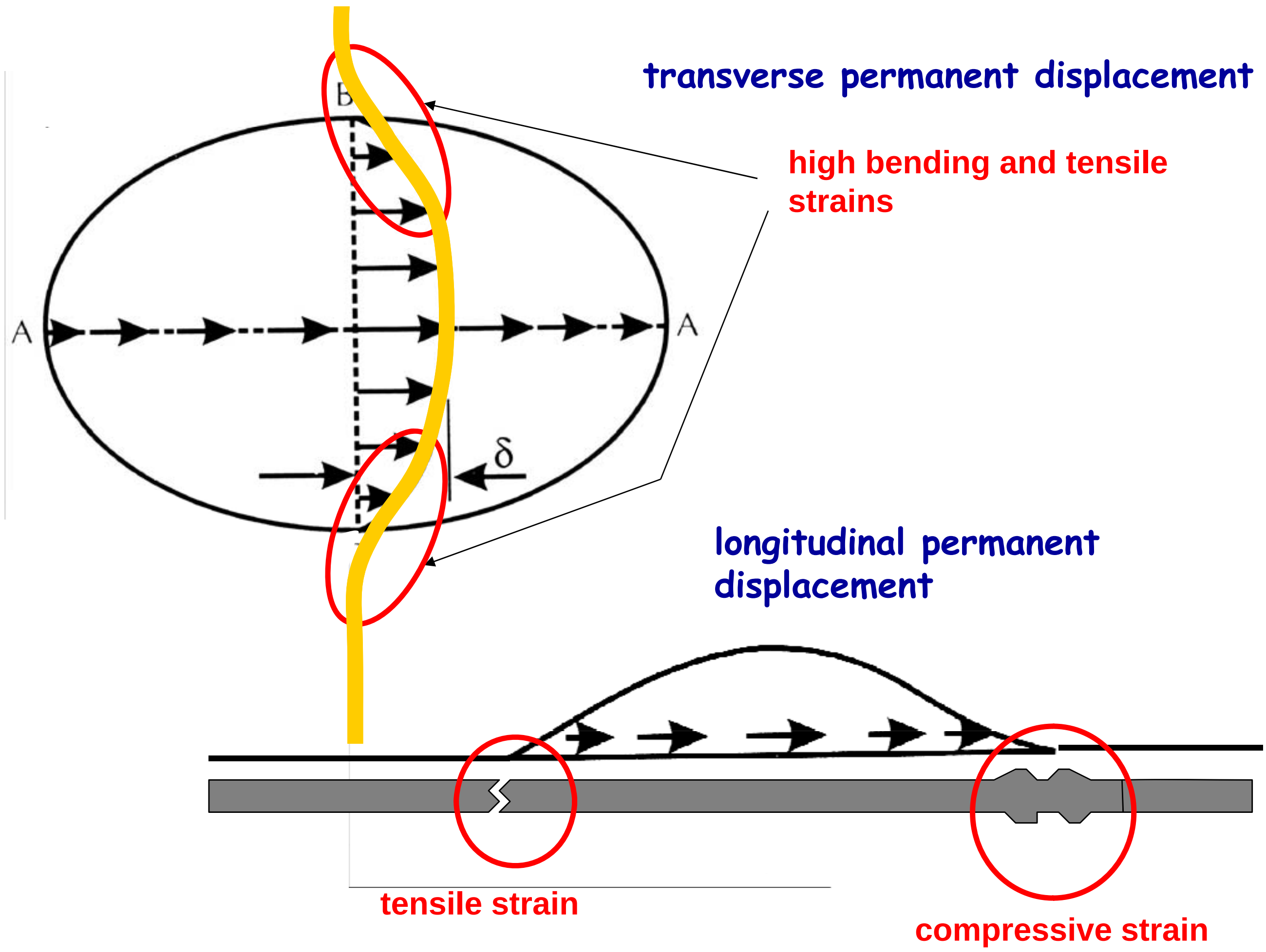
river mouth
(b)

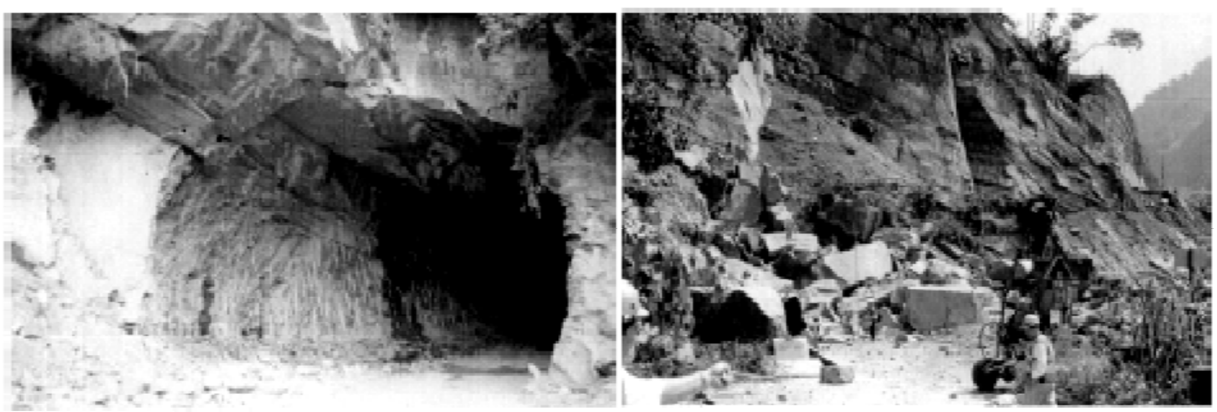
Many thousands cubic meters of soil are mobilized during slope failures, and move for distances ranging from a few centimeters to several meters...
(...when soil strength deteriorates during shaking, as for example in the case of liquefaction).

We will dwell more into the estimation of slope failure-induced displacements in one of the following lectures.

However, displacement estimates due to slope failure are not as straightforward (?) as fault-induced displacements... several seismological-geotechnical-topographical factors must be taken into account.



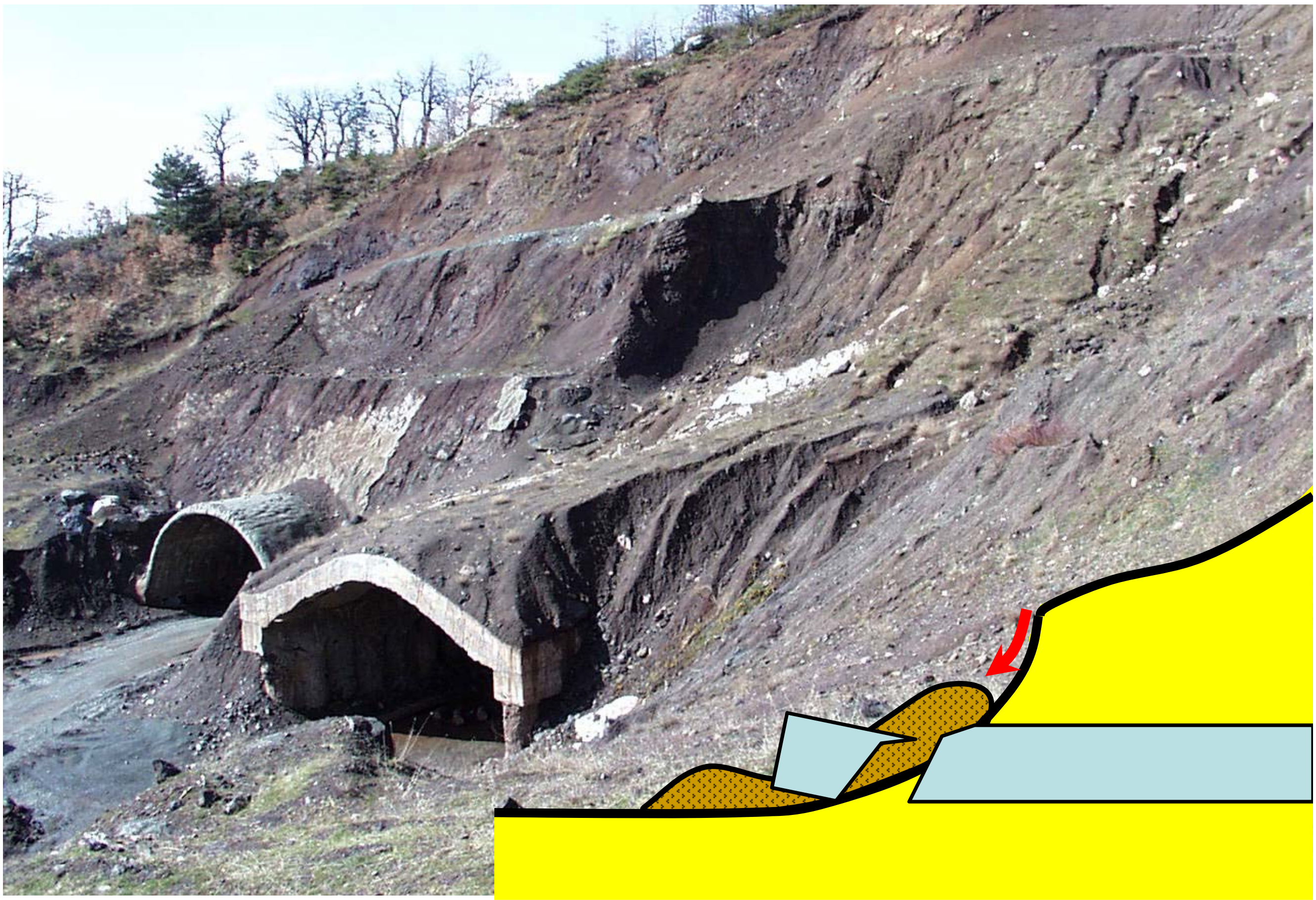




Ching-Shue tunnel before and after Chi-Chi earthquake

Failures at the portals of Ling-Leng tunnel (left) and Maa-Ling tunnel (right)



Slope failure above the western portal of the Malakassi C Tunnel.
(E. Hoek & P. Marinos, 4th Report on Egnatia Highway Project, March 1999).

Afterword

Compared to permanent displacements, **transient displacements effects are less detrimental regarding underground structure response**, since transient displacements are not only... transient, but also related to significantly smaller magnitudes.

For example, a very strong earthquake with predominant period 0.70sec and maximum ground acceleration a_{max} = 0.80g, will result in transient displacements with a magnitude in the order of few centimeters only.

$S_{max} = a_{max}\, T^2/(2\pi)^2$ = 10 cm **only**

In comparison, the permanent displacement due to the fault rupture will well exceed 1.00m

On the other hand ...

Transient displacements due to wave propagation affect the whole length of the underground structure (possibly several km), and not only the part of the structure located at the vicinity of the fault trace ($\pm$ 50m). Moreover, **transient displacements due to wave propagation have a considerably smaller "return period"** (100-500 years), compared to the return period of fault activation (10,000-100,000 years).

Both these factors highlight the importance of transient displacement effects for the seismic design of underground structures.

7. Seismic Design of Underground Structures

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Part B:
Seismic design under transient ground displacements

Oct. 2016

Basic Assumptions

- Harmonic seismic waves
- No sliding at the soil-structure interface
- kinematic soil-structure interaction effects can be ignored (flexibility factor F)
- All analytical expressions herein refer to <u>strains</u> rather than stresses.

the 3rd assumption is valid when…

$$F = \frac{2E_m(1-\nu_l^2)\left(D/2\right)^3}{E_l(1+\nu_m)t^3} > 20$$

E_m=ground's Young's modulus
E_l=structure's Young's modulus
ν_m=ground's Poisson ratio
ν_l= structure's Poisson ratio
D=cross-section diameter
t=cross-section thickness

Example: steel pipeline (e.g. natural gas pipeline)

E_m=1GPa (C_s=400m/sec)
E_l=210GPa
ν_m=0.33
ν_l= 0.2
D=1m
t=0.02m

F≅106 >>20

Example: concrete sewage pipeline

E_m=1GPa (C_s=400m/sec)
E_l=30GPa
ν_m=0.33
ν_l= 0.2
D=2.5m
t=0.1m

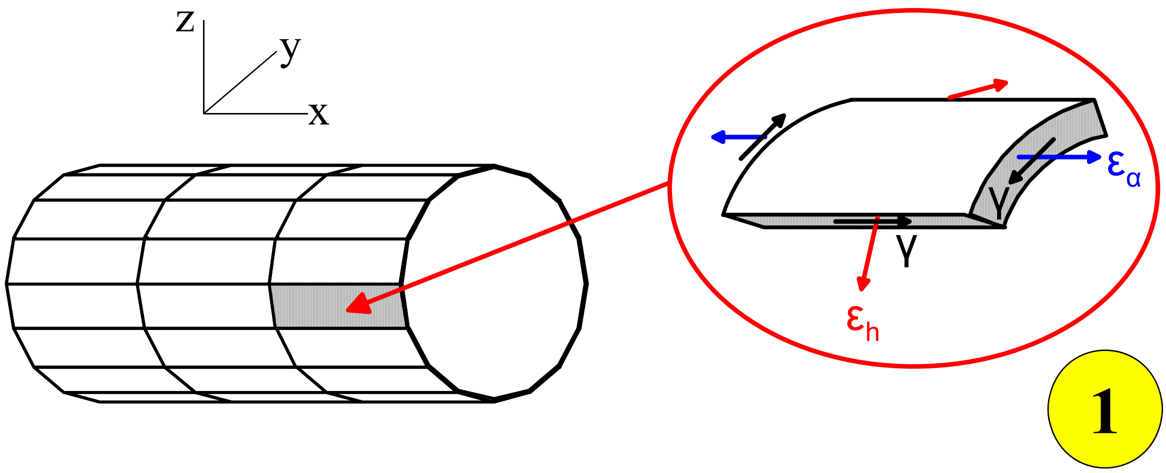
F≅95>>20

Example: concrete tunnel (e.g. Metro tunnel)

E_m=1GPa (C_s=400m/sec)
E_l=30GPa
ν_m=0.33
ν_l= 0.2
D=10m
t=0.25m

F≅385>>20

Strain state of thin-walled underground structures



A 3-D shell modeling the underground structure, when subjected to imposed displacements from the surrounding soil, will develop...

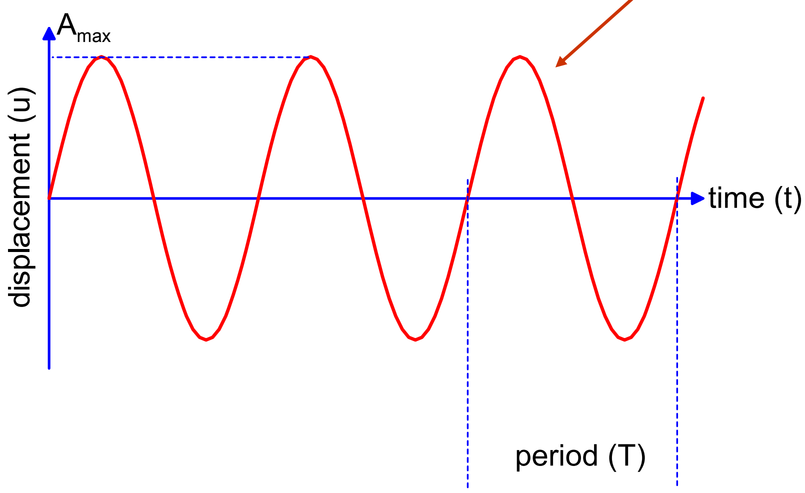
- axial strains ε_α
- in-plane shear strains γ
- hoop strains ε_h

The normal strain and the shear strains at the inside and the outside faces of the shell ***can be customarily ignored...*** **(why??)**

harmonic excitation:

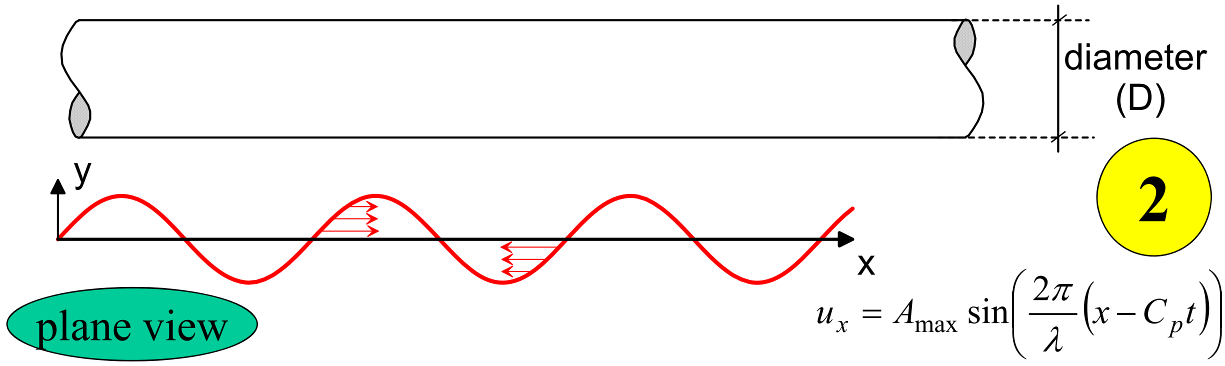
$$u^* = A_{max}\, exp\left(i\frac{2\pi}{\lambda}(x-Ct)\right)$$

$$u = Im(u^*) = A_{max}\, sin\left(\frac{2\pi}{\lambda}(x-Ct)\right)$$



$\lambda = CT =$ *wavelength* $\qquad C =$ *propagation velocity*

P-wave propagating along the axis (xy plane)



$$u_x = A_{max} \sin\left(\frac{2\pi}{\lambda}\left(x - C_p t\right)\right)$$

ground strain= structure strain:

strain amplitude

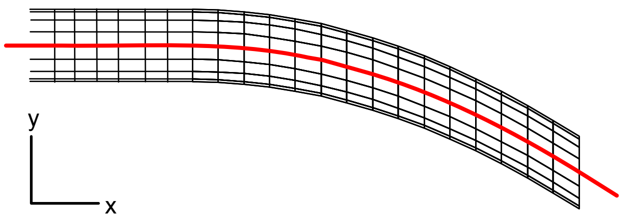
$$\varepsilon_a = \varepsilon_x = \frac{\partial u_x}{\partial x} = \frac{2\pi A_{max}}{\lambda} \cos\left(\frac{2\pi}{\lambda}\left(x - C_p t\right)\right)$$

thus…

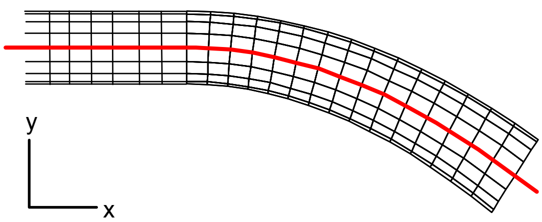
$$\varepsilon_{a,max} = \frac{2\pi A_{max}}{\lambda} = \frac{V_{max}}{C_p}$$

maximum seismic velocity

S-wave propagating along the axis (xy plane)



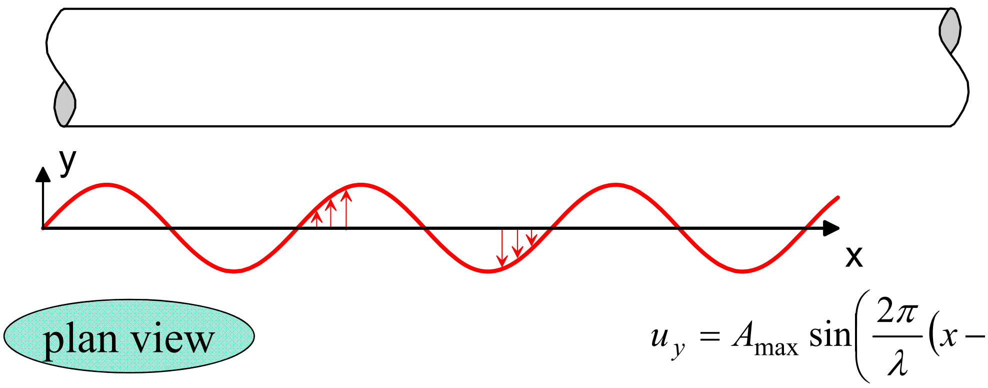
No-slip assumption at the soil-structure interface →“shear beam” model
No slip suggests zero relative displacements between “cross-sections”, thus zero bending strains.



A **full-slip** assumption at the soil-structure interface would suggest that only bending strains develop, and that shear strains are zero
(“bending beam” model)

In the real world, some slippage will always occur, especially during strong excitations
The conservative no-slip assumption is adopted for design purposes

adopting the "shear beam model"…



$$u_y = A_{\max} \sin\left(\frac{2\pi}{\lambda}(x - C_s t)\right)$$

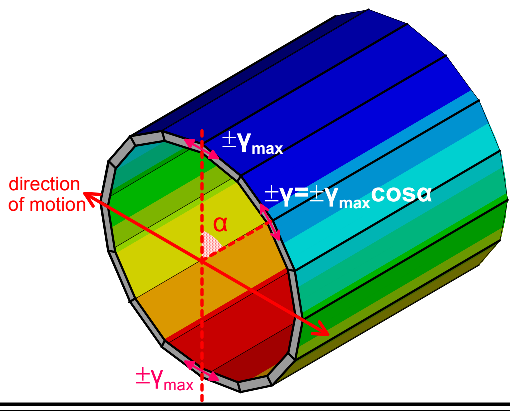
---

ground strain = structure strain:

$$\gamma = \frac{\partial u_y}{\partial x} = \frac{V_{\max}}{C_s} \cos\left(\frac{2\pi}{\lambda}(x - C_s t)\right)$$

$$\gamma_{max} = \frac{V_{max}}{C_S}$$

strain distribution along the cross-section



strains on a 3-D shell (except axial strain) do not retain a constant value along the cross-section. Strain distribution is a function of the polar angle α and, of course, time

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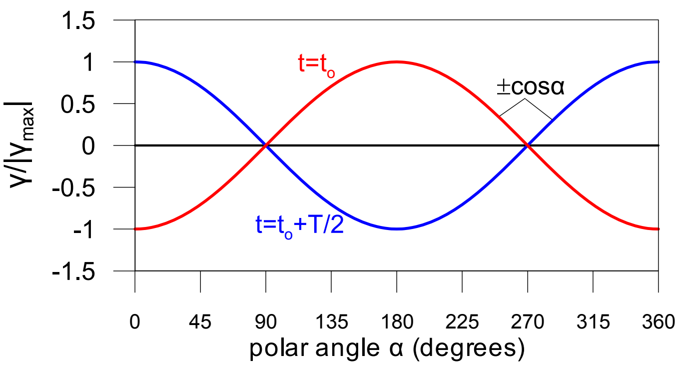
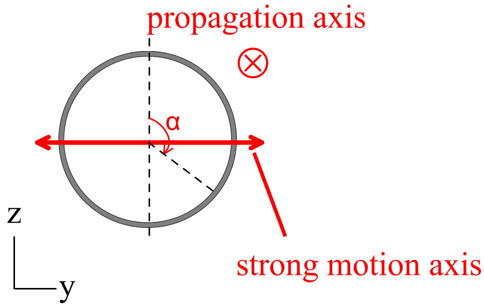
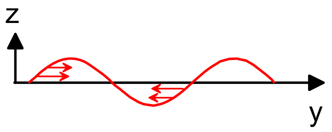


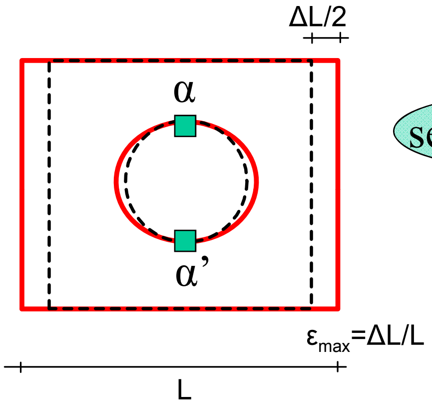


angle α is measured clockwise, starting from z-axis

Transverse P-wave (yz plane)



$$u_y = A_{max} \sin\left(\frac{2\pi}{\lambda}\left(y - C_p t\right)\right)$$

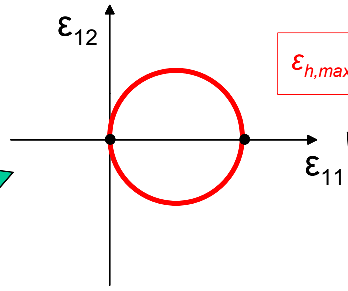


section view

3

ground strain:

$$\varepsilon_y = \frac{\partial u_y}{\partial y}$$

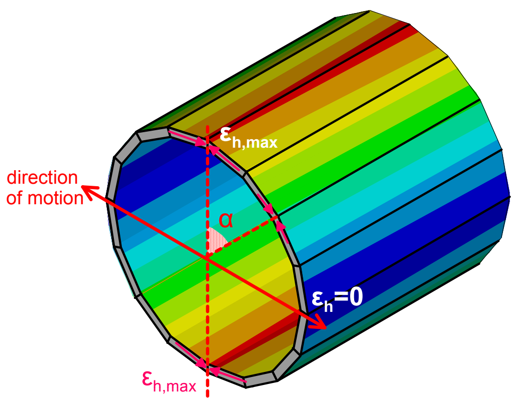


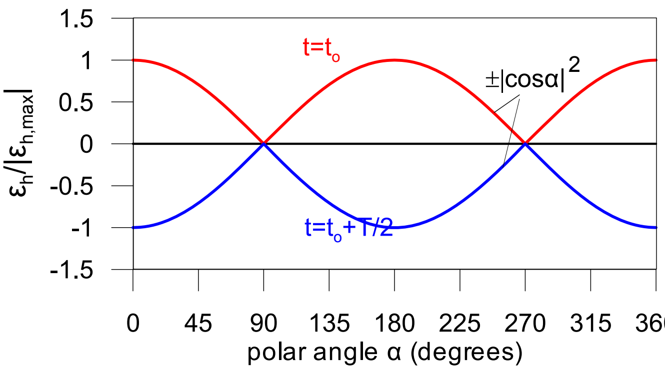
$\varepsilon_{h,max} = \pm\varepsilon_{max}$

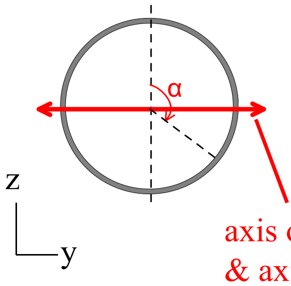
structure strain at points α & α':

$$\boldsymbol{\varepsilon_h = \varepsilon_y = \frac{V_{max}}{C_p} cos\left(\frac{2\pi}{\lambda}\left(y - C_p t\right)\right)}$$

strain distribution along the cross-section



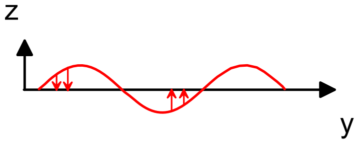




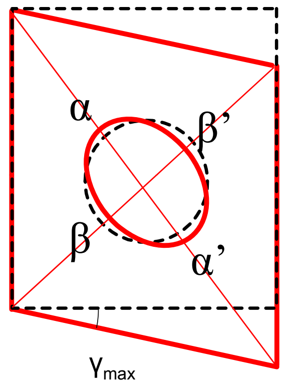
axis of propagation & axis of strong motion

angle α is measured clockwise, starting from z-axis

Transverse S-wave (yz-plane)



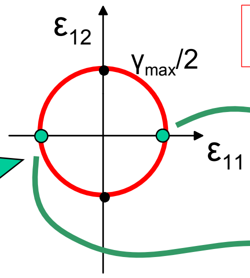
$$u_z = A_{\max} \sin\left(\frac{2\pi}{\lambda}(y - C_s t)\right)$$



section view

ground strain:

$$\gamma_{yz} = \frac{\partial u_z}{\partial y}$$



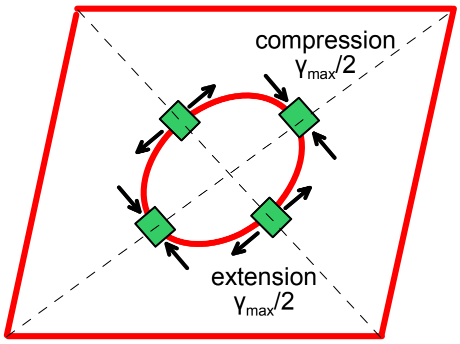
$\varepsilon_{h,max} = \gamma_{max}/2$

structure strain:

$$\varepsilon_{h,\alpha\&\alpha'} = \gamma_{yz}/2 = \frac{V_{max}}{2C_s} \cos\left(\frac{2\pi}{\lambda}(y - C_s t)\right)$$

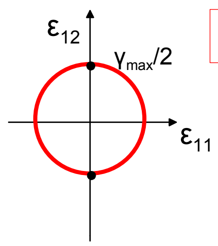
$$\varepsilon_{h,\beta\&\beta'} = -\gamma_{yz}/2 = \frac{V_{max}}{2C_s} \cos\left(\frac{2\pi}{\lambda}(y - C_s t)\right)$$

strain distribution along the cross-section:

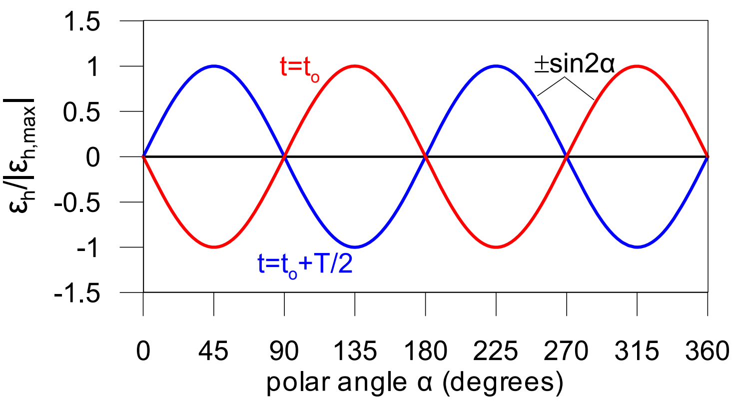


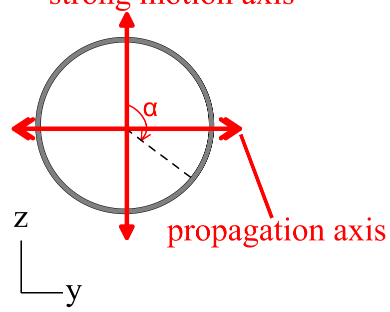
Principal strains (tensile and compressive) are located at a plane inclined by $\pm 45^o$

(why??)

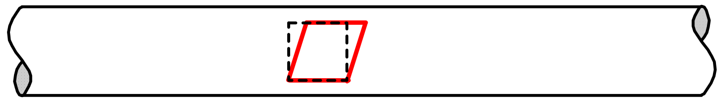


$\varepsilon_{h,max} = \gamma_{max}/2$



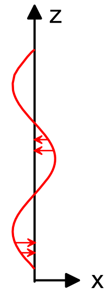


Transverse S-wave (xz-plane)



side view

direction of propagation

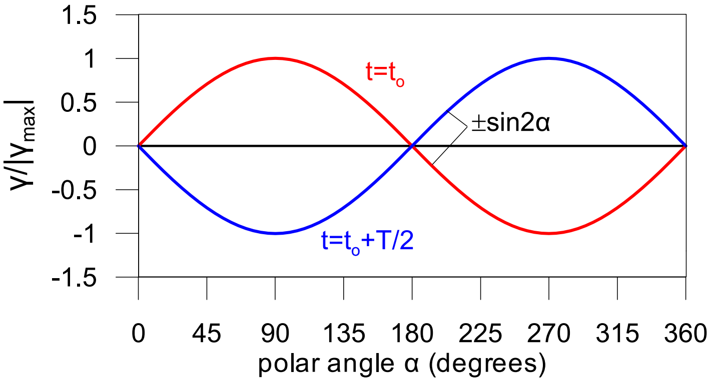


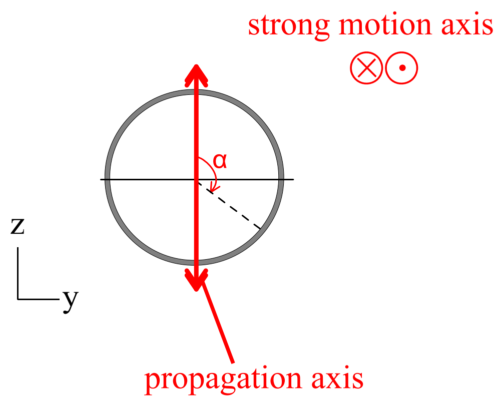
$$u_x = A_{\max} \sin\left(\frac{2\pi}{\lambda}(z - C_s t)\right)$$

ground strain= structure strain:

$$\gamma = \frac{\partial u_x}{\partial z} = \frac{V_{\max}}{C_s} \cos\left(\frac{2\pi}{\lambda}(z - C_s t)\right)$$

strain distribution along the cross-section





Transverse S-wave propagation results in shear structure strains

(as in the case of S-wave propagation along the axis, at xy-plane)

but with different strain distribution along the cross-section

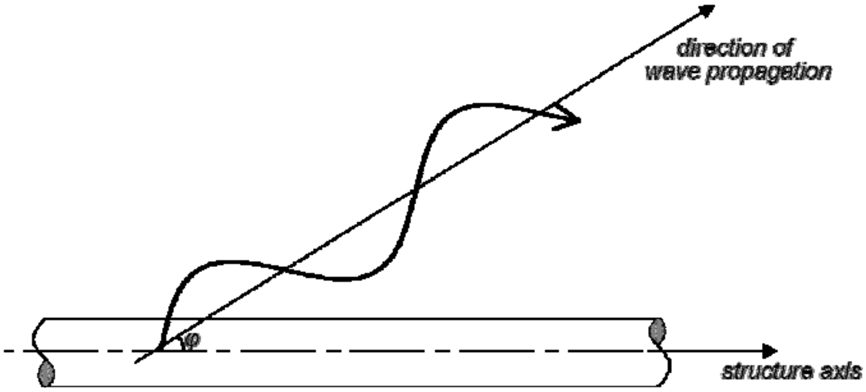
Summarizing…

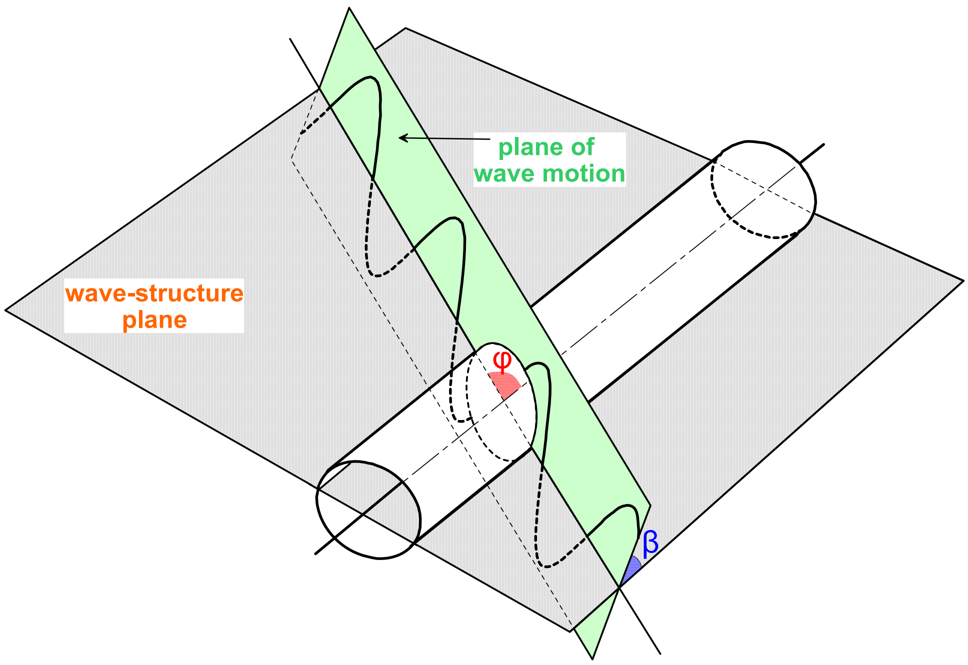
Case	$\varepsilon_{\alpha,max}$	γ_{max}	$\varepsilon_{h,max}$
P-wave along the axis (xy)	V_{max}/C_p		
S-wave along the axis (xy)		V_{max}/C_s	
Transverse P-wave (yz)			V_{max}/C_p
Transverse S-wave (yz)			$V_{max}/2C_s$
Transverse S-wave (xz)		V_{max}/C_s	

In the real world…

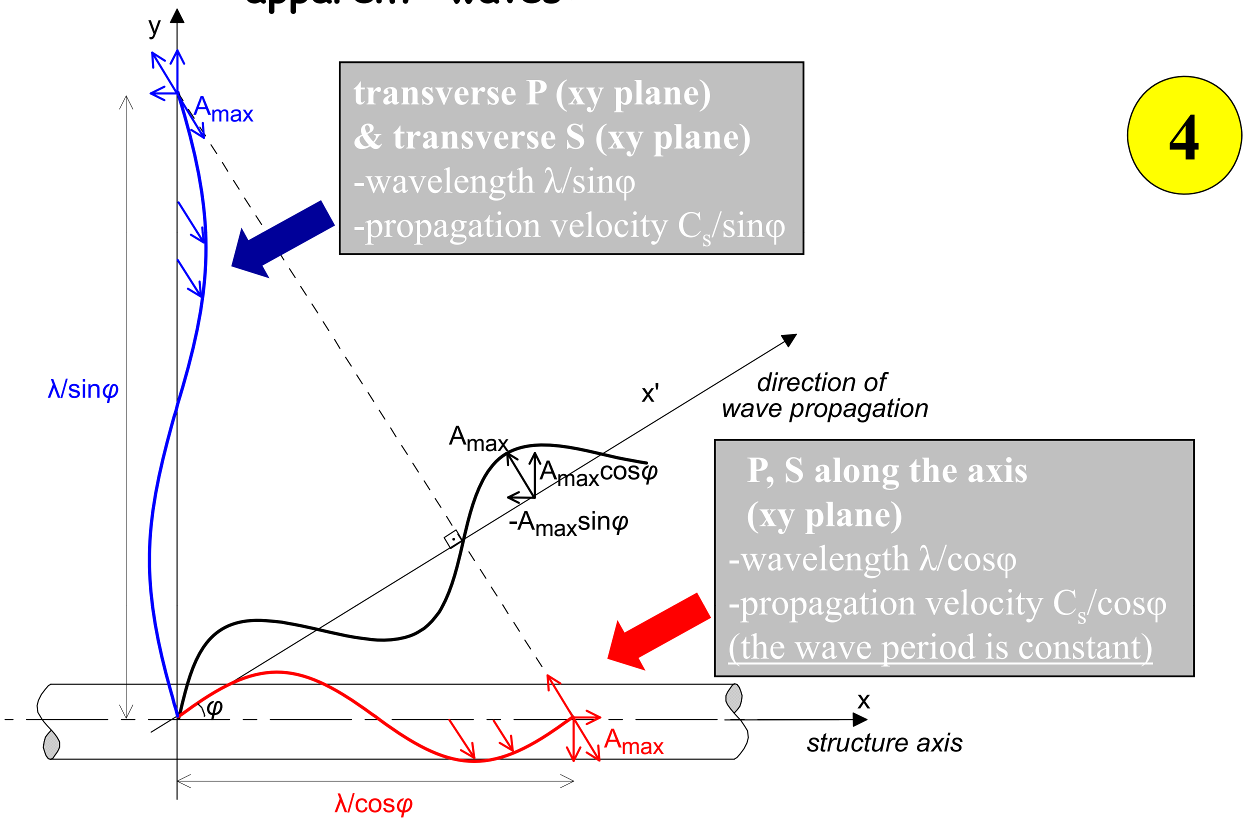
a seismic wave will cross the axis of the structure under a random angle φ, measured at the plane defined by the structure axis and the wave propagation axis.

Orientation of this plane at the 3-D space is random *(there is no practical way to predict it or define it)*





An SV-wave (the direction of motion coincides with the plane of propagation xy) can be analyzed into 4 "apparent" waves:



In order to estimate maximum design strains for this generic case, we must consider:

a) superposition of strains (distribution along the cross-section) that result from each "apparent" wave propagation.

b) maximization of the resulting expressions for strains, to eliminate the unknown angles **φ** & **β**

(angle β is the angle formed by the plane of strong motion and the plane of propagation, and is defined for S-waves only)

Implementing this procedure for S- and P-waves propagating at a random angle relatively to the structure axis, results in the following maximum design strains...

maximum strains	S-wave (normalized over the V_{max}/C_s ratio)	P-wave (normalized over the V_{max}/C_p ratio)
axial ε_α	0.50	1.00
shear γ	1.00	1.00
hoop ε_h	0.50	1.00
von Misses ε_{vM}	$0.87/(1+\nu_l)$	$1.00/(1+\nu_l)$
major principal ε_1	0.71	1.00
minor principal ε_3	-0.71	-1.00

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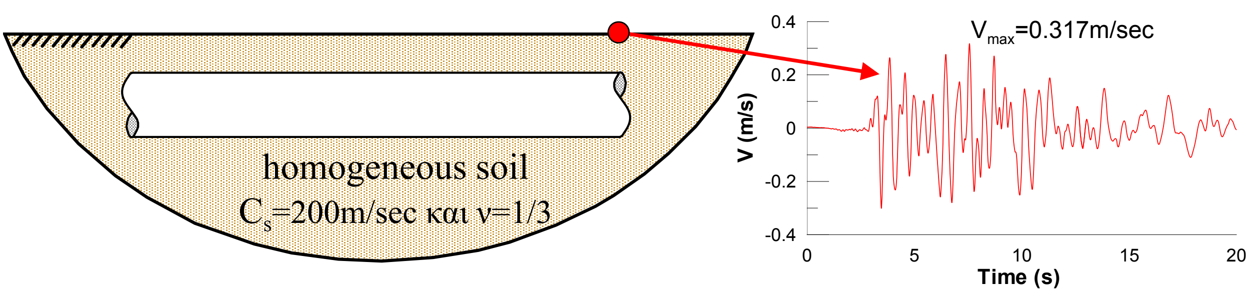
ATTENTION: Strain components do not attain these maximum values <u>at the same position along the cross-section</u>. Thus, simply adding them to derive von Misses and principal strains is an over-conservative approach.

Homework...

Leukada 16/8/03 earthquake



Verify the structural integrity of the following structures for P & S waves:

<u>steel pipeline</u>

- D=1.5m, t=0.015m
- $\varepsilon_{all}^{compression}$=40t/d (%)<5% (EC8)
- $\varepsilon_{all}^{extension}$ =2% for the main body
  =0.5% for peripheral butt welds

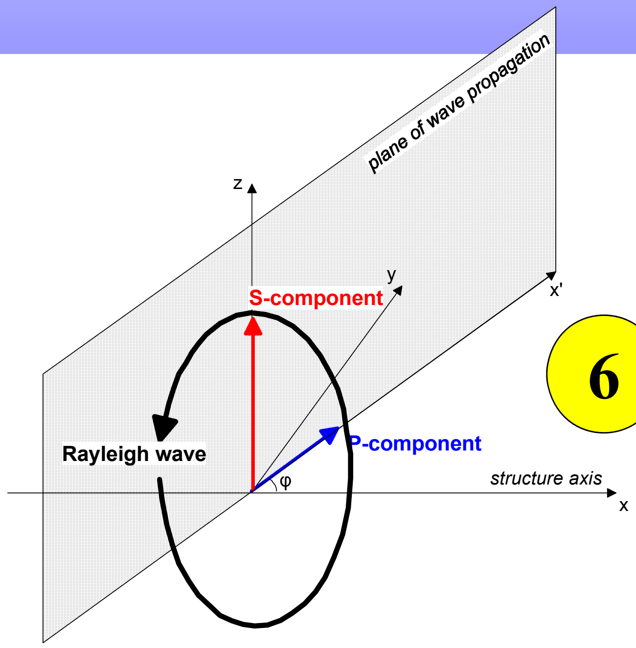
<u>concrete tunnel</u>

- D=10m, t=0.20m
- $\varepsilon_{all}^{compression}$=0.35%
- $\varepsilon_{all}^{extension}$=2%

Justify the expressions that you apply for the estimation of seismic strains!

In the real world…

Underground structures are located relatively close to the ground surface, and are subjected to surface Rayleigh wave effects too (*e.g. at valleys, near slopes, at large distances from rupture)*



A Rayleigh waves is equivalent to a P-wave and a SV-wave, propagating simultaneously and featuring a **phase lag equal to π/2**

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ATTENTION: We should not just… add strains for a P-wave and a SV-wave propagating at the same angle φ in order to find the maximum strains, as **maxima do not occur at the same time.**

Maximum <u>(after proper superposition)</u> design strains for a Rayleigh wave propagating at a random angle relatively to the structure axis:

maximum strain	**R-wave** (normalized over the $V_{max,V}/C_R$ ratio)
axial ε_α	0.68
shear γ	1
hoop ε_h	0.68
von Misses ε_{vM}	$0.86/(1+\nu_l)$
major principal ε_1	0.68
minor principal ε_3	-0.68

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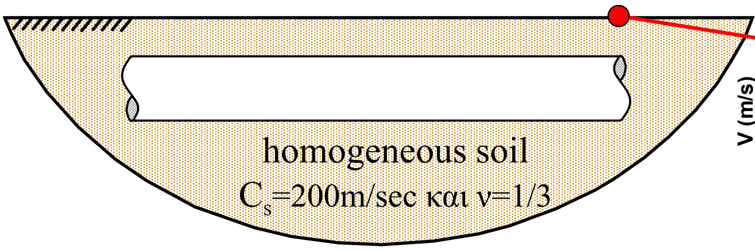
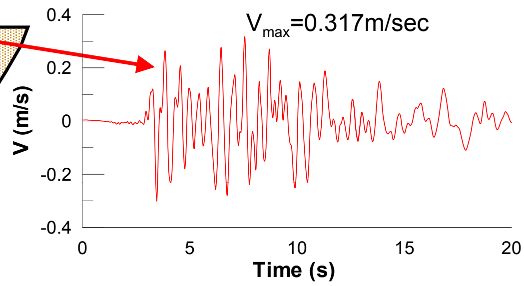
Note:
Rayleigh wave propagation velocity $C_R \cong 0.94\ C_s$ is estimated at a depth $z=1.0\lambda_R$

It is worth trying to verify the above expressions for Rayleigh waves at home!

Homework (continued)...



Leukada 16/8/03 earthquake



Verify the structural integrity of the following structures for R waves:

<u>steel pipeline</u>

- D=1.5m, t=0.015m
- $\varepsilon_{all}^{compression}$=40t/d (%)<5% (EC8)
- $\varepsilon_{all}^{extension}$ =2% for the main body
  =0.5% for peripheral butt welds

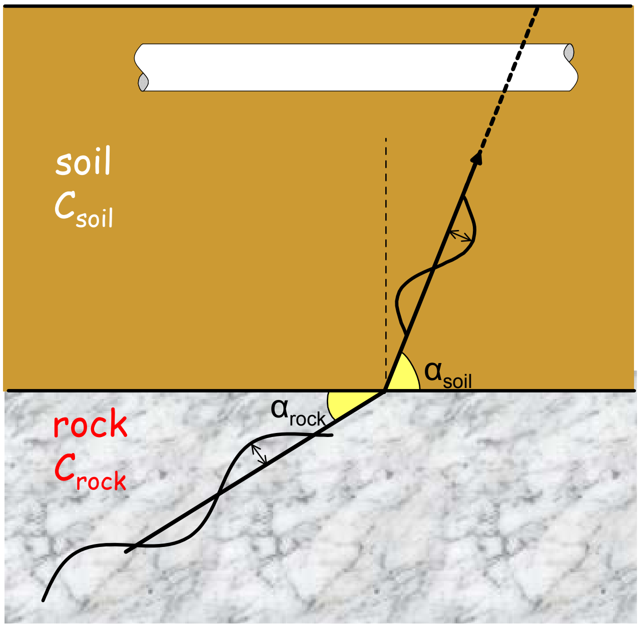
<u>concrete tunnel</u>

- D=10m, t=0.20m
- $\varepsilon_{all}^{compression}$=0.35%
- $\varepsilon_{all}^{extension}$=2%

Justify the expressions that you apply for the estimation of seismic strains!

In the real world…

Consider the case of an underground structure located in a soft soil layer, overlying the bedrock



Seismic waves propagate to the soil-bedrock interface at a random angle α_{rock}, refract, and continue to the soft soil layer

according to Snell's law:

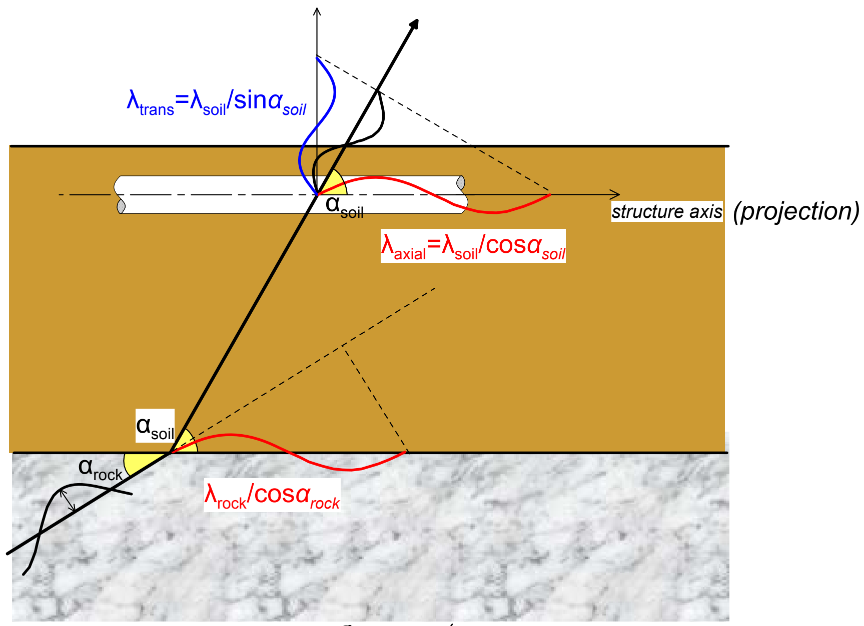
$$\cos\alpha_{soil} = \cos\alpha_{rock}\frac{C_{soil}}{C_{rock}}$$

the predominant period of the refracted wave is not altered, thus…

$$\lambda_{soil} = \lambda_{rock}\frac{C_{soil}}{C_{rock}}$$

The refracted wave, propagating at an angle α_{soil} relatively to the structure, can be analyzed (as before) into apparent waves…

A. a horizontal apparent wave with a propagation velocity $C_{soil}/\cos\alpha_{soil}$ …



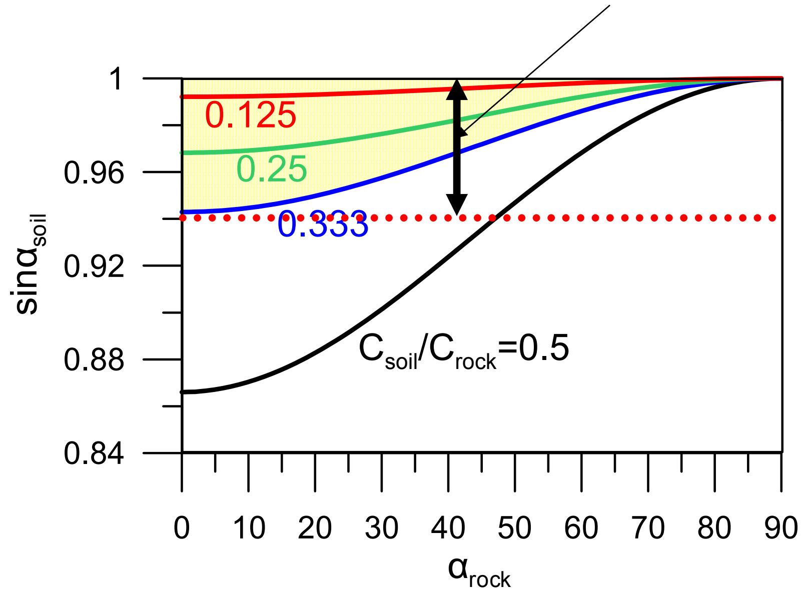
and wavelength:
$$\lambda_{axial} = \frac{\lambda_{soil}}{\cos\alpha_{soil}} = \frac{\lambda_{rock}\dfrac{C_{soil}}{C_{rock}}}{\cos\alpha_{rock}\dfrac{C_{soil}}{C_{rock}}} \Rightarrow \boxed{\lambda_{axial} = \frac{\lambda_{rock}}{\cos\alpha_{rock}}}$$

The horizontal apparent wave is equivalent to the apparent wave propagating **along the soil-bedrock interface**.

B. a vertical apparent wave with a propagation velocity $C_{soil}/sin\alpha_{soil}$ …

on the other hand:

$$sin\alpha_{soil} = \sqrt{1-\cos^2\alpha_{soil}} = \sqrt{1-\cos^2\alpha_{rock}\left(\frac{C_{soil}}{C_{rock}}\right)^2}$$



So, the "apparent" propagation velocity of the vertical apparent wave is practically equal to the **wave propagation velocity in soft soil C_{soil}** (and $\Lambda_{trans}=\Lambda_{soil}=C_{soil}\ T$)

An extra, unknown parameter must be considered here, compared to the homogeneous rockmass case- **the angle α_{rock}**

Design strains are estimated via the superposition of strains along the cross-section, and the subsequent maximization of the resulting expressions for the unknown angles φ, β και α_{rock} …

Normalized strains $\dfrac{\varepsilon}{V_{max}/C_{soil}}$	**S-wave** ($C=C_S$)	**P-wave** ($C=C_P$)
axial ε_α	**$0.50C_{soil}/C_{rock}$**	**$0.3C_{soil}/C_{rock}$**
shear γ	**$0.43C_{soil}/C_{rock}+0.98$**	**$2.0C_{soil}/C_{rock}$**
hoop ε_h	**$0.36C_{soil}/C_{rock}+0.50$**	**$0.5C_{soil}/C_{rock}+1.0$**
von Misses ε_{vM}	**$(0.38C_{soil}/C_{rock}+0.85)/(1+\nu_l)$**	**$(0.58C_{soil}/C_{rock}+1.0)/(1+\nu_l)$**
major principal ε_1	**$0.5C_{soil}/C_{rock}+0.5$**	**$0.63C_{soil}/C_{rock}+1.0$**
minor principal ε_3	**$-0.5C_{soil}/C_{rock}-0.5$**	**$-0.63C_{soil}/C_{rock}-1.0$**

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The above expressions are valid for $C_{soil}/C_{rock} < 0.35$.
In the opposite case, these expressions will be over-conservative, and the use of the corresponding expressions for homogeneous rockmass conditions is proposed.

Homework (continued)...

Repeat the previous homework assignment for the case where the the structure is constructed near the surface of a homogeneous soft soil layer ($C_{S,SOIL}$=200m/s), overlying the marl bedrock ($C_{S,ROCK}$=700m/s).

Comparison to ALA-ASCE (2001) recommendations

ALA-ASCE 2001 $\varepsilon_a = V_{max}/aC$

V_{max}= peak ground velocity generated by ground shaking

C = apparent propagation velocity for seismic waves, conservatively (?) assumed to be equal to 2000 m/s.

a = 1 for P and R waves, 2 for S waves

Strains	S-wave ($C=C_S$)	P-wave ($C=C_P$)
axial ε_α	$0.50 \cdot V_{max}/C_{rock}$	$0.3 \cdot V_{max}/C_{rock}$
shear γ	$(0.98+0.43 \cdot C_{soil}/C_{rock}) \cdot V_{max}/C_{soil}$	$2.00 \cdot V_{max}/C_{rock}$
hoop ε_h	$(0.50+0.36 \cdot C_{soil}/C_{rock}) \cdot V_{max}/C_{soil}$	$(1.00+0.50 \cdot C_{soil}/C_{rock}) \cdot V_{max}/C_{soil}$

Comparison to ALA-ASCE (2001) recommendations

input data:
C_{soil}=200m/sec
C_{rock}=2000m/sec (*bedrock*)
$C_p=2C_s$
V_{max}=75cm/sec

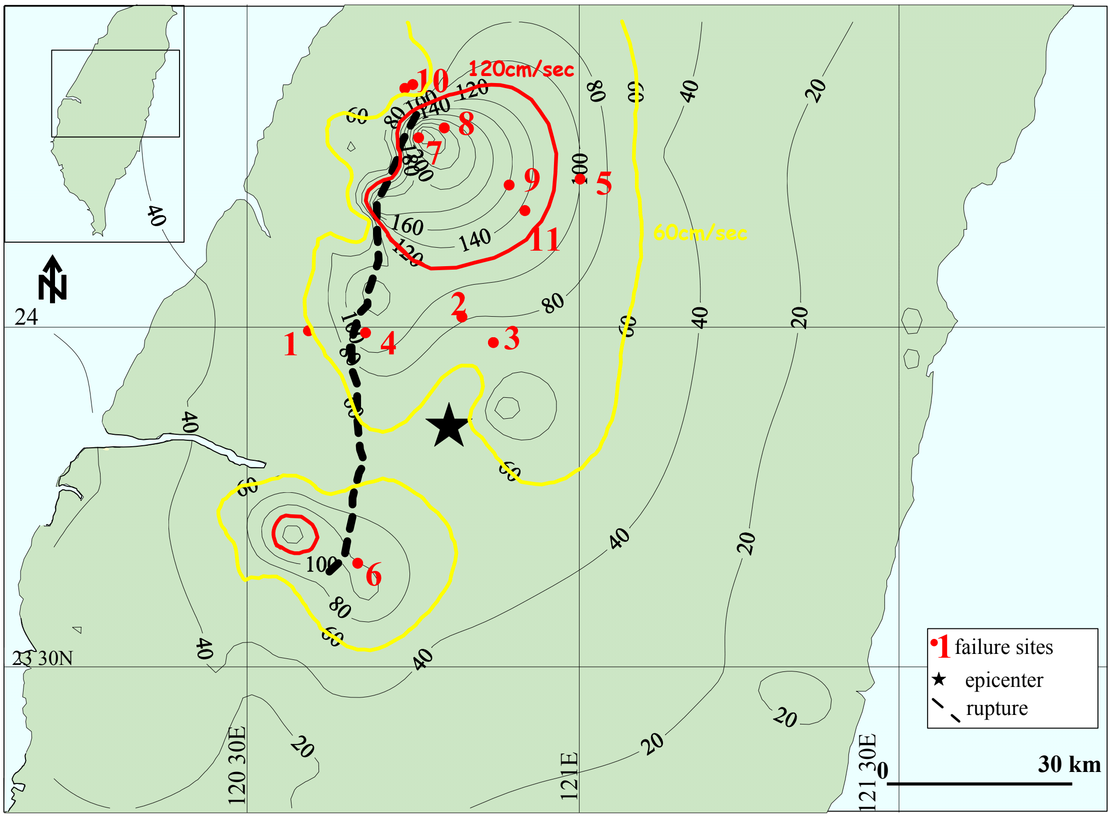
Strains	Homogeneous rock		Soft soil over rock		Rayleigh wave
	S-wave	P-wave	S-wave	P-wave	
axial ε_α	0.019%	0.019%	0.019%	0.005%	0.255%
shear γ	0.037%	0.019%	0.383%	0.037%	0.375%
hoop ε_h	0.019%	0.019%	0.201%	0.196%	0.255%

ALA-ASCE 2001

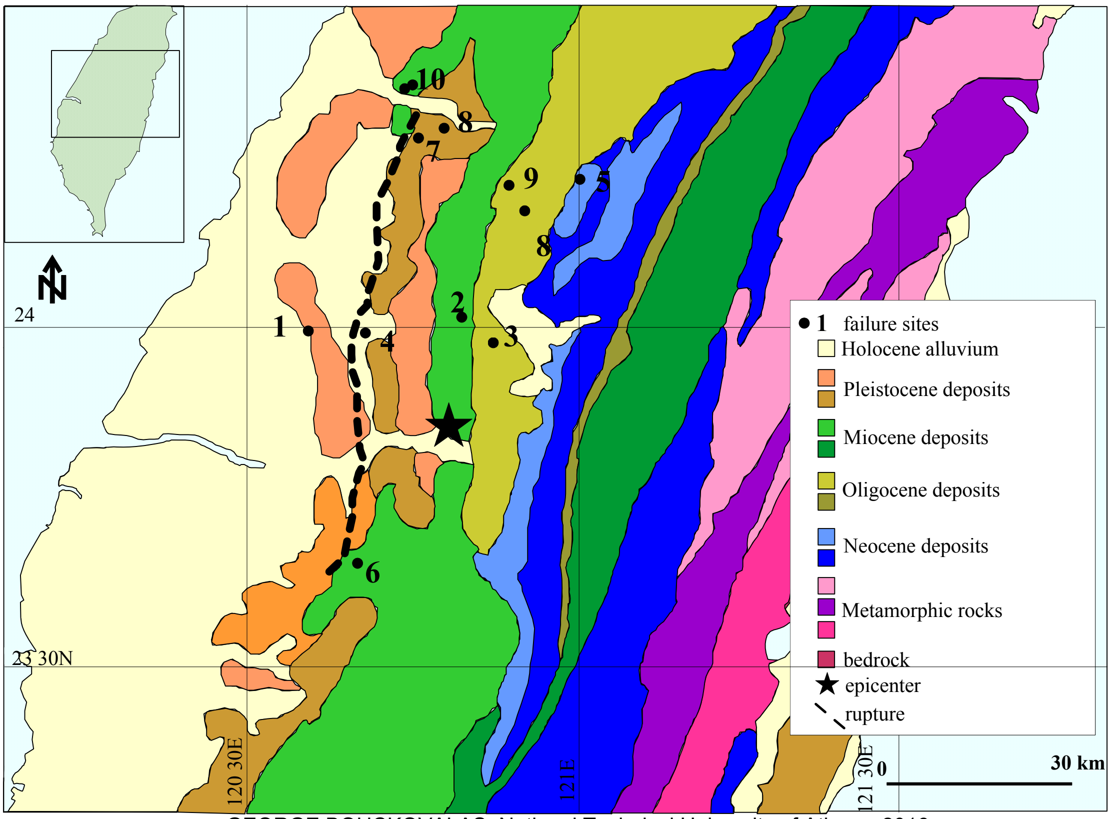
$\varepsilon_a = V_{max}/aC$ =0.037% (conservatively a=1)

major discrepancy...!

Case studies of underground structures failures during Kobe & Chi-Chi earthquakes



Case studies of underground structures failures during Kobe & Chi-Chi earthquakes

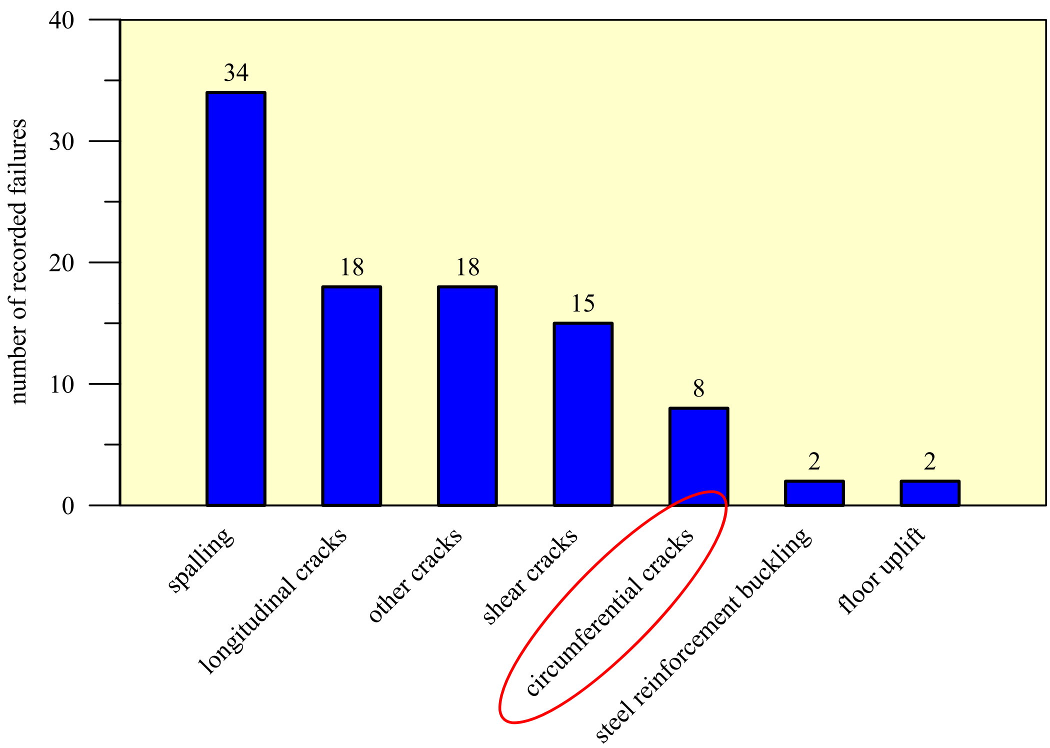


Soil classification

NEHRP (1997)	C_s (m/sec)	This study	C_s (m/sec)
A *Hard rock*	C_s>1500	A *Granite bedrock*	2000
B *Rocks*	760<C_s<1500	B1 *Cretaceous rocks (igneous and metamorphic rocks, limestone, solid volcanic deposits)*	1200
		B2 *Stiff soils and soft rocks (Pleio-Pleistocence sandstones, conglomerates, schists, marls)*	850
C *Very stiff soils/soft rocks*	360< C_s <760	C *Pleistocene clayey deposits, sands and gravels*	550
D *Stiff soils*	180< C_s <360	D *Holocence alluvium (sand, gravels, clay) and man-made deposits*	250
E *Soft soils*	C_s<180		

«soft soil»

Frequency of failure types



«Failure criterion»

development of cracks wider than w>0.2mm

crack width is related to the stress applied on the steel reinforcement bars
(Gergely and Lutz, 1968, ACI Committee 224, 1995)

$$\boxed{w = 0.076 \cdot \beta \cdot f_s \cdot \sqrt[3]{d_c A}}$$

for a typical tunnel... $f_{s,lim}$=140MPa

failure when:
$\varepsilon_1 * E_{steel} > f_{s,lim}$

stresses higher than $f_{s,lim}$ suggest larger crack widths, or concrete spalling

«Κριτήριο αστοχίας»

εμφάνιση ρωγμών πλάτους w>0.2mm

εύρος ρωγμής (w, σε in/1000) συναρτήσει της τάσης στον οπλισμό:
(Gergely and Lutz, 1968, ACI Committee 224, 1995)

$$\boxed{w = 0.076 \cdot \beta \cdot f_s \cdot \sqrt[3]{d_c A}}$$

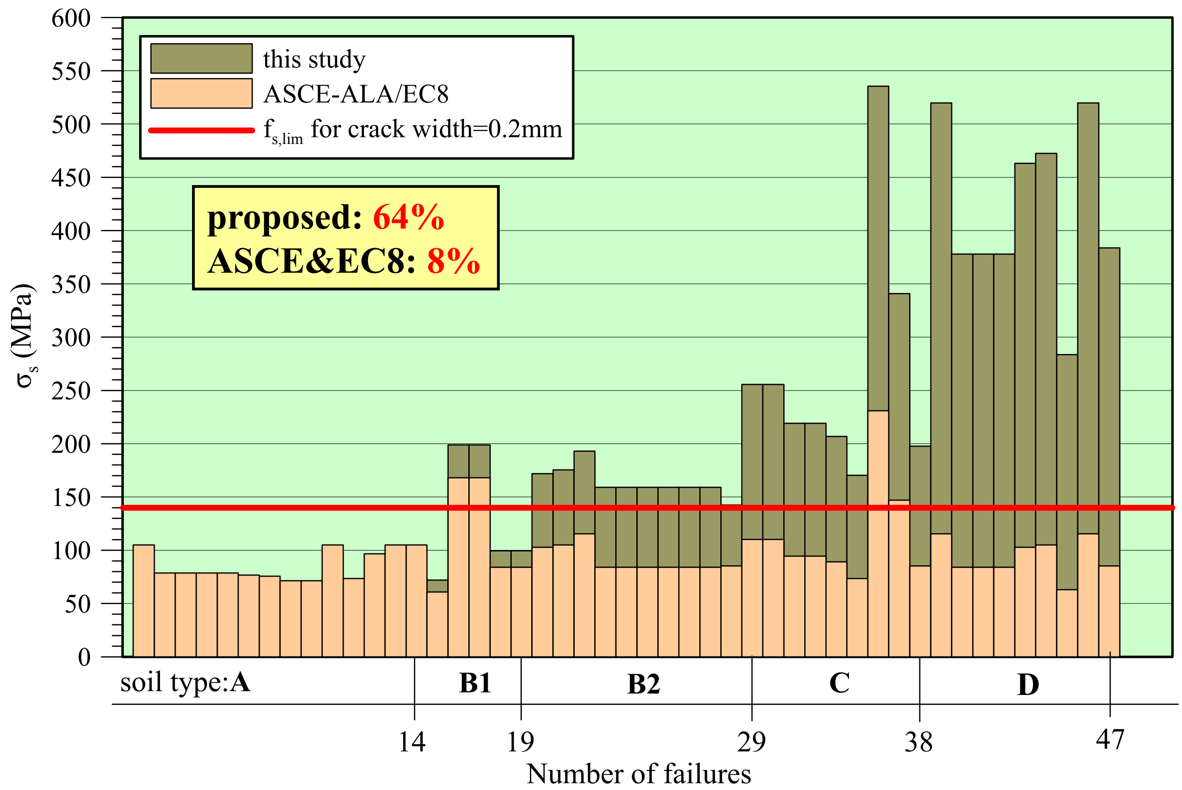
f_s : η εφελκυστική τάση του χάλυβα οπλισμού (σε ksi)
d_c: το πάχος της επικάλυψης σκυροδέματος (σε in)
A: το εμβαδόν του σκυροδέματος που περιβάλλει κάθε ράβδο οπλισμού
(εμβαδόν εφελκυόμενης περιοχής της διατομής/αριθμός ράβδων, σε in^2)
β: διορθωτικός συντελεστής

για τυπική σήραγγα $f_{s,lim}$=140MPa, $d_c \approx$7cm, A ≈200cm^2, β=1.2

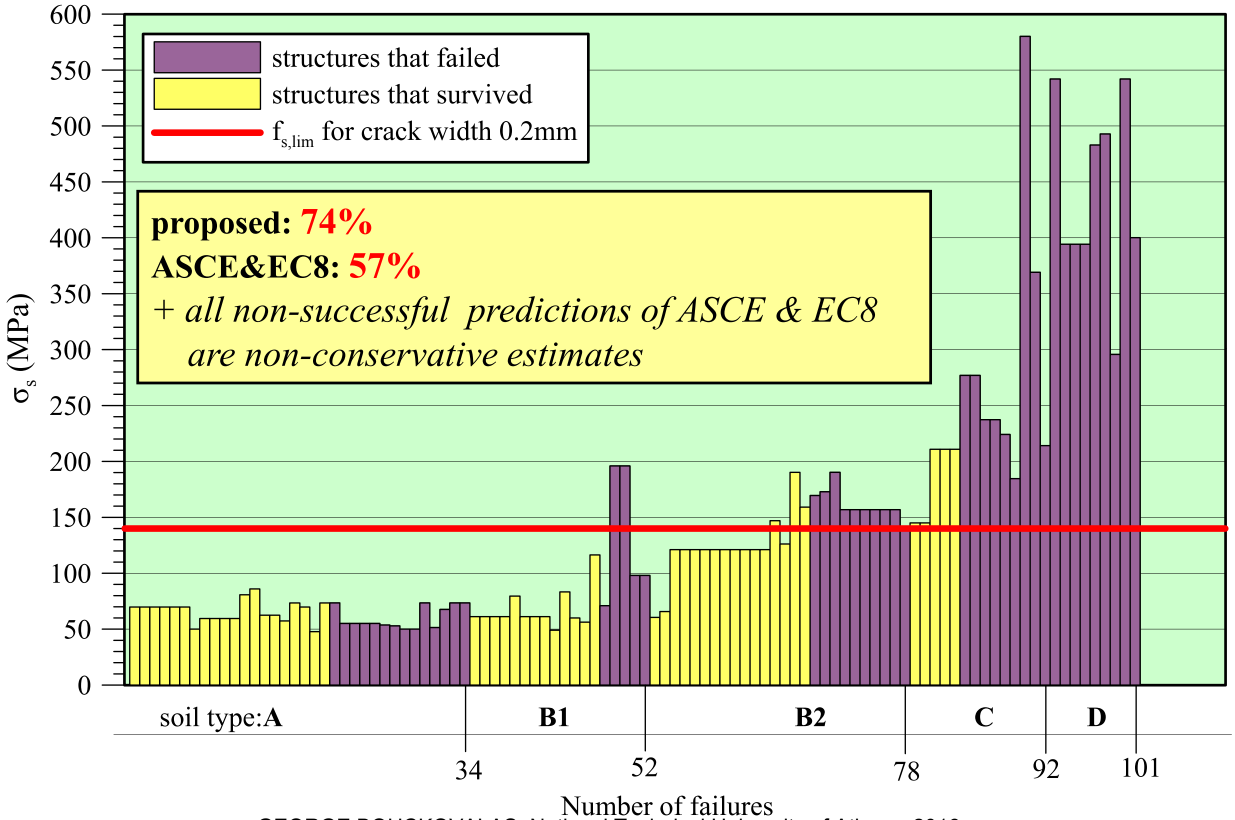
πρόβλεψη αστοχίας:
$\varepsilon_1 * E_{steel} > f_{s,lim}$

τάσεις μεγαλύτερες της $f_{s,lim}$ σημαίνουν ρωγμές μεγαλύτερου εύρους ή/και εμφάνιση θρυμματισμού

a-posteriori failure prediction



Prediction of actual structure response



Afterword…

The presented analytical expressions are valid for **flexible** (F>20), and **“infinitely long”** underground structures. The stress state becomes more complicated in areas of bends and T-ees. However, analytical methodologies have been proposed for such cases too, and provide relatively accurate results. When the underground structure is constructed by discrete, jointed pieces, the flexibility of the joints must also be taken into account in the assessment of its response.

Generally speaking, when the in-situ conditions diverge significantly from the discussed assumptions, more elaborate **numerical analysis tools** must be applied for the seismic design of the underground structure. The structure is modeled as a beam or a shell, soil-structure interaction is simulated via elasto-plastic springs, and the seismic excitation is applied at the base of the springs that are used to model soil response.

An extensive presentation of such numerical methodologies is beyond the scope of this lecture. Their basic principles are however presented in the following **case study,** regarding the numerical stress analysis of a crude oil steel pipeline, due to the possible activation of a normal and a strike slip fault crossing its route **(permanent ground displacements)**

7. Seismic Design of Underground Structures

Ch. Gantes
Professor N.T.U.A.

G. Bouckovalas
Prosessor N.T.U.A.

(Part C)

Octomber 2016

A. Earthquakes and Underground Structures

B. Seismic design against "transient" displacements

C. Seismic design against "permanent" displacements

C.J. Gantes, G.D. Boukovalas
Department of Civil Engineering
National Technical University of Athens

ANALYSIS OF BURIED PIPELINES AT FAULT CROSSINGS

Contents of presentation

- Pipeline Geometry, Material Properties & Construction Techniques
- Failure Criteria
- Downthrows at Seismic Faults
- Pipeline Modeling - Fault Movement Modeling
- Beam & Mixed Model Results - Influence of Internal Pressure
- Conclusions

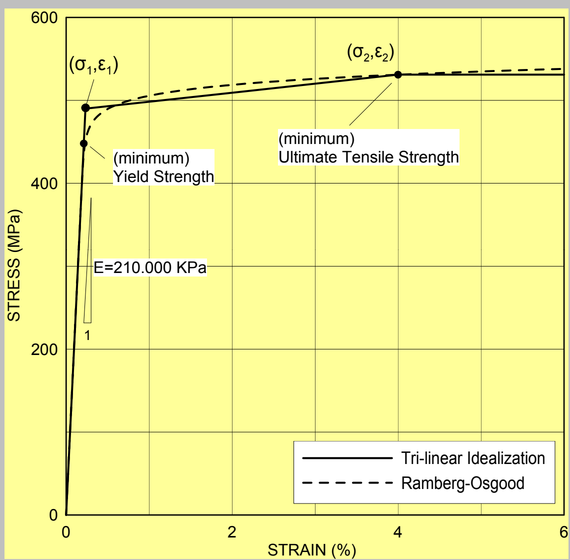
Geometry, material properties and construction techniques

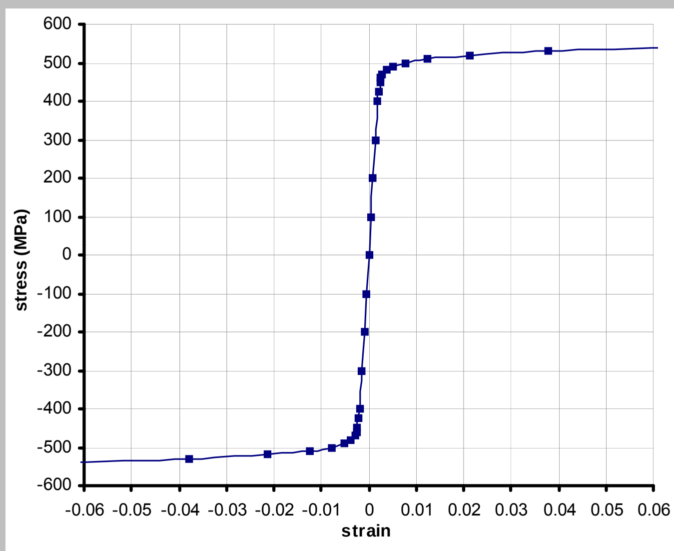
At the specific locations of active faults crossings, heavy wall NPS 16 line pipes with a wall thickness of 8.74mm will be installed, made of API-5LX60 steel.

The nominal backfill cover varies from 0.60m in rocky areas to 0.90m in cross-country areas and 1.20m under major roads.

The pipeline steel is modeled with a typical API-5LX60 tri-linear stress-strain curve based on the provisions of ASCE.

Pipeline material properties





Failure criteria

Pipelines resist the imposed displacements mainly through axial (tensile or compressive) strains, thus it is more meaningful to talk in terms of strains than stresses.

Considered Failure Criteria

1. Limiting compressive strain to avoid elastic or plastic buckle, associated with local wrinkling.

$$e^{*c} = 0.84 - 0.0035\frac{D}{t}$$

For an NPS 16 in. x 8.74mm pipe the former expression yields 0.677%.

Failure criteria

2. Limiting the tensile strain for girth welds due to metallurgical alterations induced to the heat-affected zone during the welding process.

The allowable tensile strain for butt (peripheral) welding is conservatively taken as 5‰. The latter criterion is used throughout the analysis.

Section Conclusions

- Relative ground movements caused by fault rupture are displacement-controlled
- The limiting strain is taken equal to 5‰

Downthrows at seismic faults

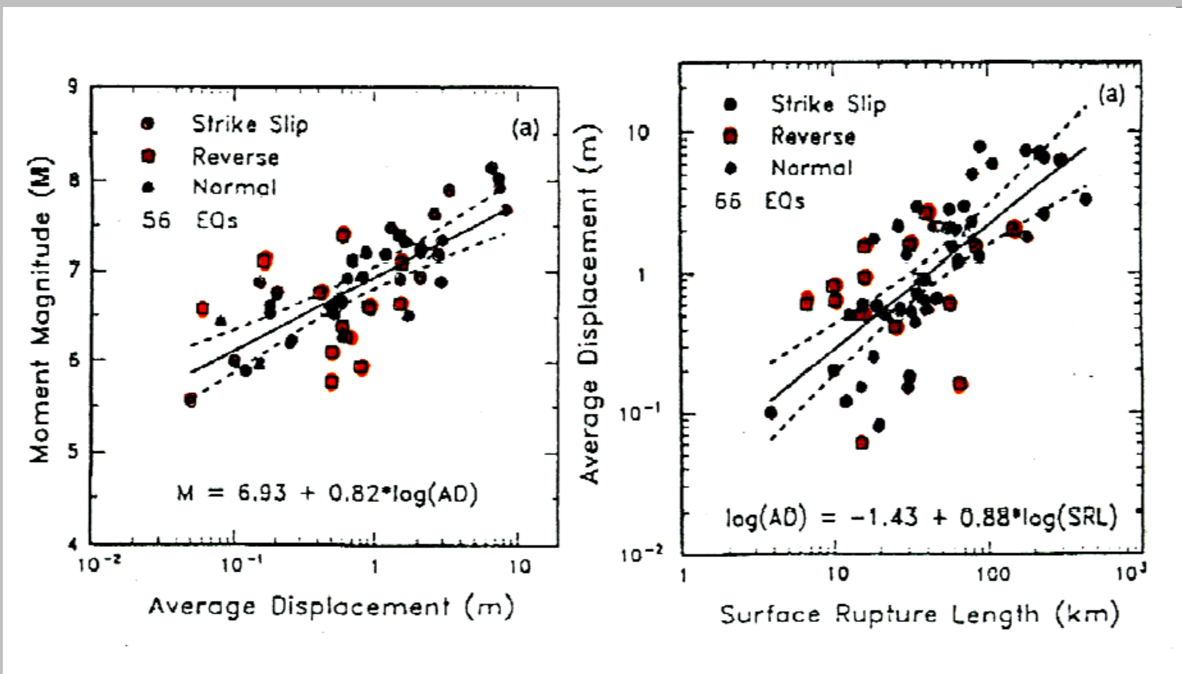
Two fault types are considered:

- •Normal fault
- •Strike slip fault

The critical crossings are identified with reference to:

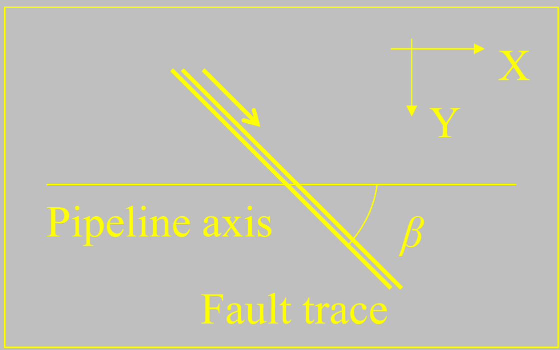
- •Anticipated ground movements
- •Geology at the area of crossing
- •Angle of intersection between fault trace and pipeline axis

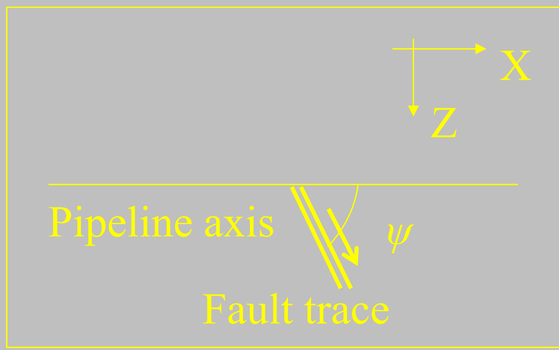
Εμπειρικές σχέσεις υπολογισμού της ΜΕΣΗΣ αναμενόμενης μετατόπισης *λόγω τεκτονικών διαρρήξεων Wells & Coppersmith, 1994). Οι* ***ΜΕΓΙΣΤΕΣ*** *αναμενόμενες μετατοπίσεις είναι περίπου διπλάσιες.*



Normal fault

Fault geometry





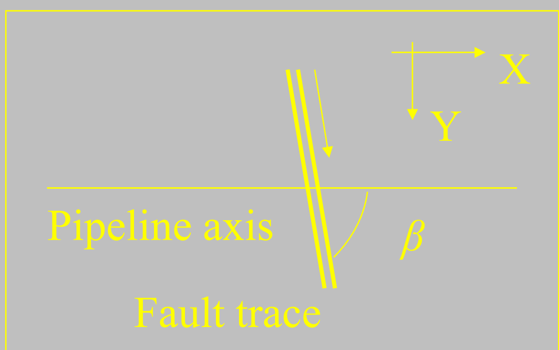
Anticipated downthrow DZ = 30 cm

$$DX = DZ \cdot \cot(\psi) \cdot \cos(\beta) \quad DY = DZ \cdot \cot(\psi) \cdot \sin(\beta)$$

For β=45° and ψ = 63° DX=DY=0.36DZ

Strike slip fault

Fault geometry



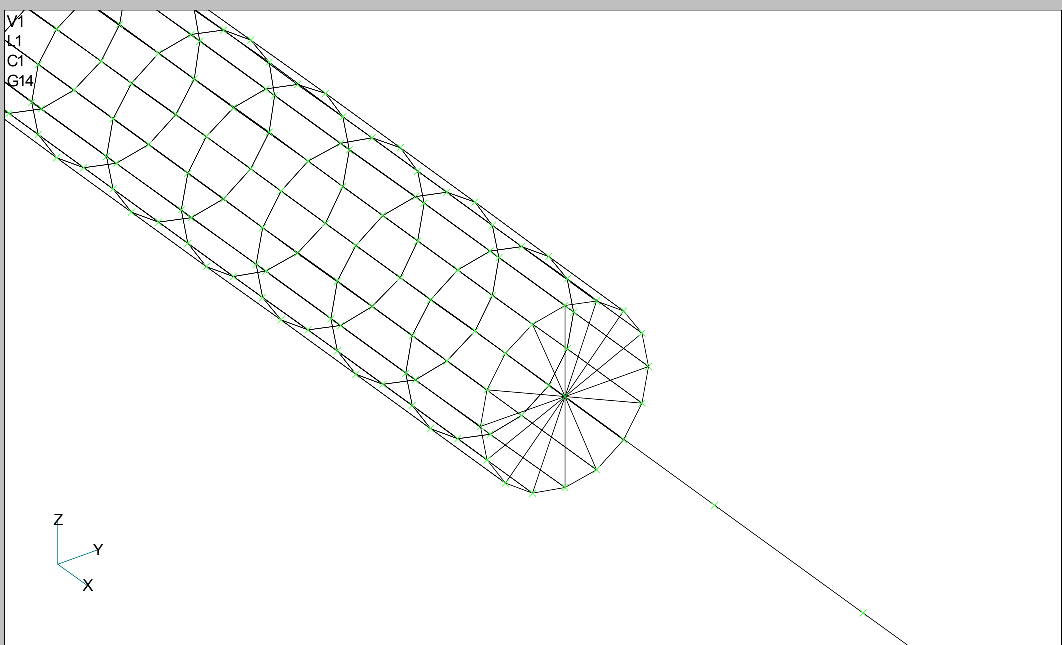
Anticipated left lateral Slip S = 30 cm, β=80°

$$DX = S \cdot \cos(\beta) = 0.17S \quad DY = S \cdot \sin(\beta) = 0.98S$$

Pipeline modeling

- Non linear FE Analysis for material behavior and large deformations are considered using NASTRAN.
- Two models, a beam (BM) and a mixed beam-shell model (MM) are used for the analysis.
- A straight pipeline segment of length 1200m is considered for both models and the fault rupture is applied in the middle.
  - The Beam Model (BM) implements 3D beam elements, having the mechanical properties of a tube with 16" diameter and 8.74mm thickness made of API-5LX60 steel.
  - The Mixed Model (MM) combines shell elements near the expected fault to capture stress concentrations, and 3D beam elements further away from the fault, where low stresses are expected. Coupling of shell and the beam part of the model is done with the use of rigid elements.

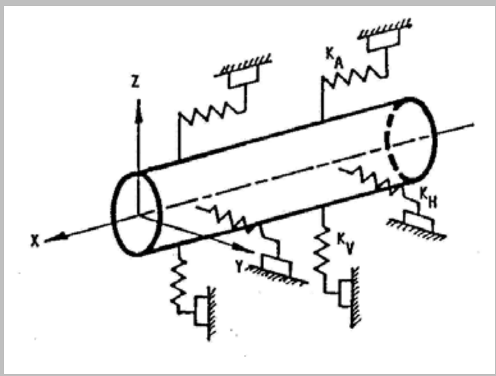
Pipeline modeling



Coupling of shell and beam elements

Modeling of soil around buried pipelines

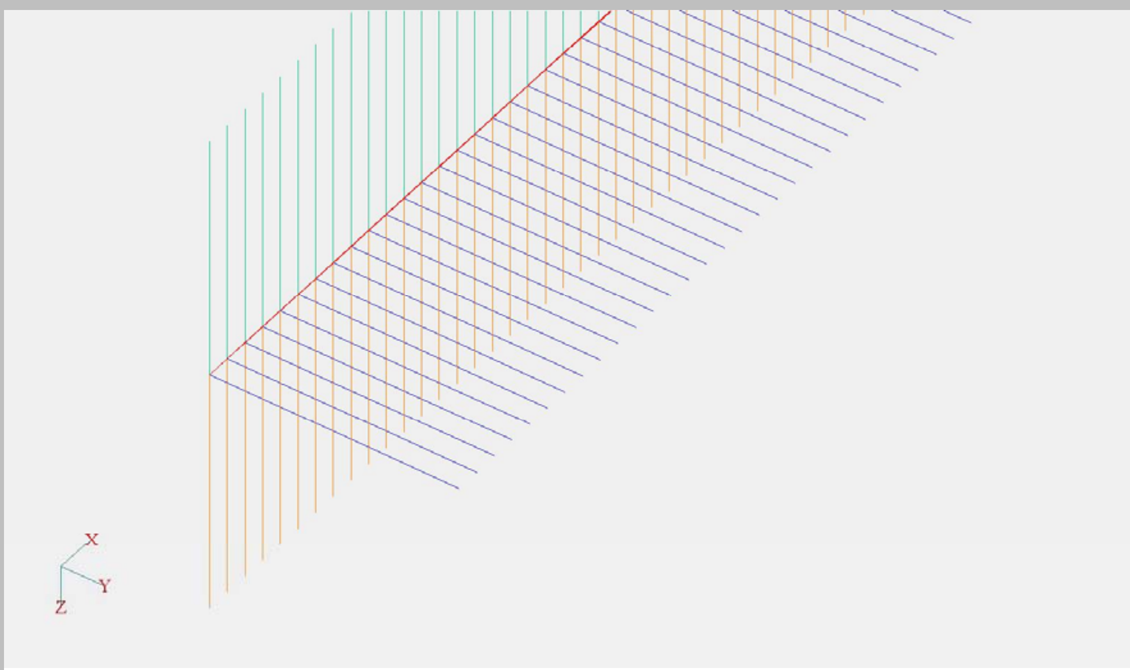
The soil is modeled with four sets of inelastic springs, two in the local X-Y plane and two in the global vertical Z direction where different upward and downward reactions occur



The soil springs are computed assuming cohesionless materials (sands) such as the backfill soil used along the pipeline route

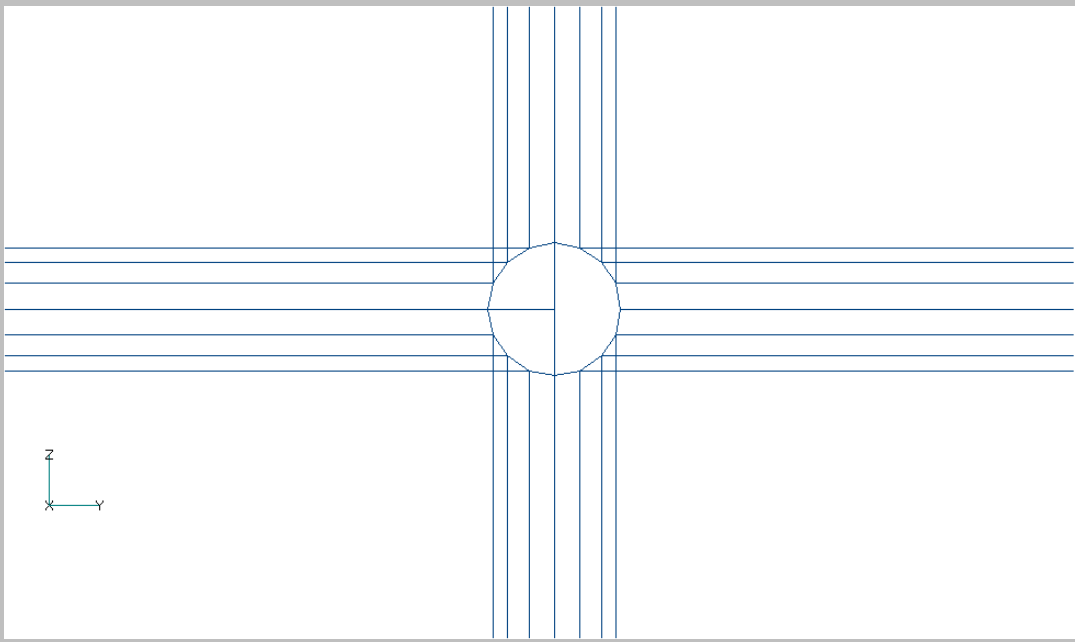
Properties of the springs are calculated according to ALA-ASCE (2005) guidelines for the seismic design of buried pipelines (for sand trench backfill)

Pipeline modeling



Transverse soil springs in the beam part of the model

Pipeline modeling



Transverse soil springs in the shell part of the model

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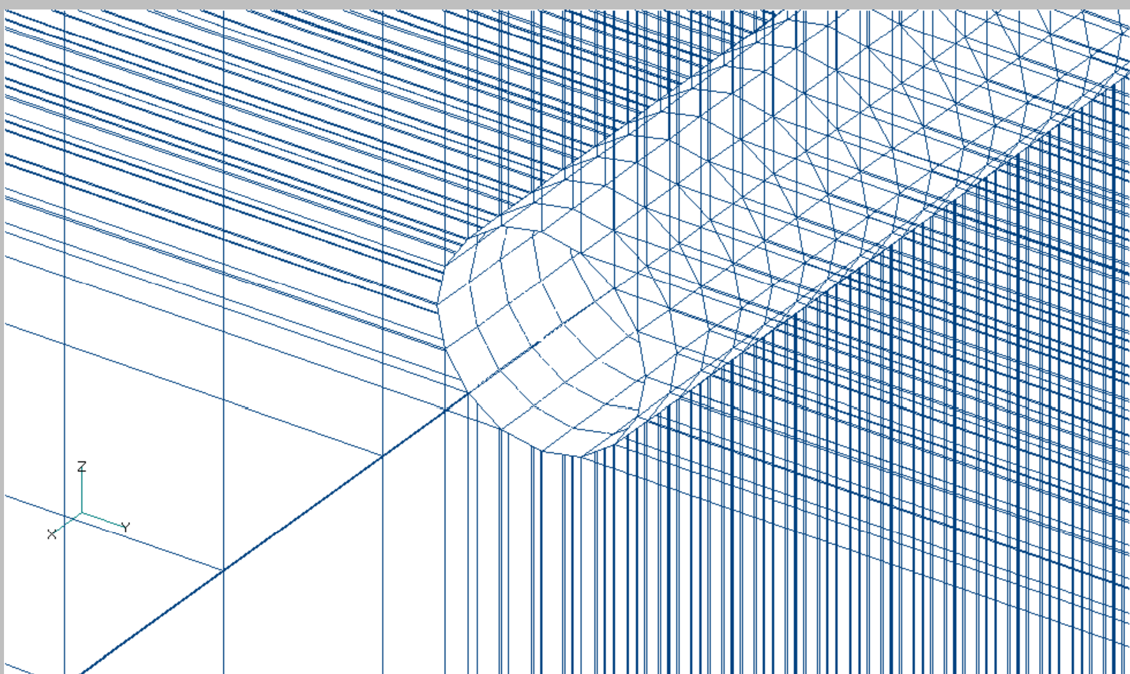
Pipeline modeling



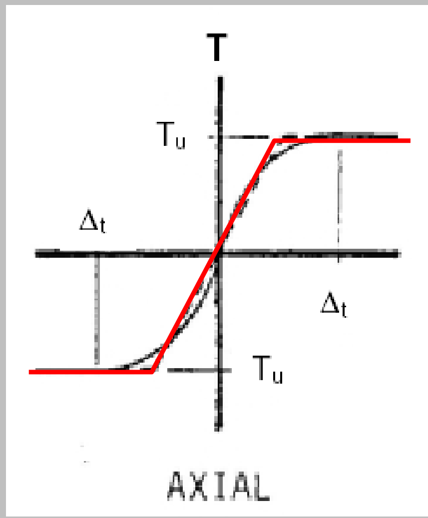
Transverse soil springs in the shell part of the model

Axial soil springs

Maximum axial soil force per unit length of the pipeline

$$T_u = \pi D H \bar{\gamma}\left(\frac{K_0 + 1}{2}\right)\tan\delta$$

$\delta \approx \frac{2}{3}\phi$ for a steel pipeline



Δ_t = 3mm (dense sand)
5mm (loose sand)

[sum of interface shear (friction) forces along the perimeter of the pipeline]

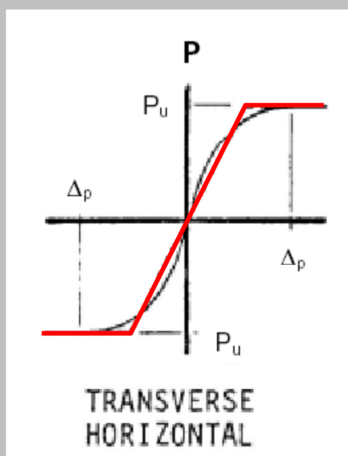
Transverse horizontal soil springs

These springs simulate the resistance from the surrounding soils to any horizontal translation of the pipeline. Thus, the mechanisms of soil-pipeline interaction are similar to those of vertical anchor plates or footings moving horizontally relative to the surrounding soils and thus mobilizing a passive type of earth pressure.

Relationship between force p per unit length and horizontal displacement y

$$p = \frac{y}{A + B \cdot y}$$

$A = 0.15\, y_u/p_u$
$B = 0.85/p_u$
$p_u = \gamma\, H\, N_{qh}\, D$
N_{qh}=horizontal bearing capacity factor
y_u = 0.07 to 0.10 (H+D/2) for loose sand
0.02 to 0.03 (H+D/2) for dense sand

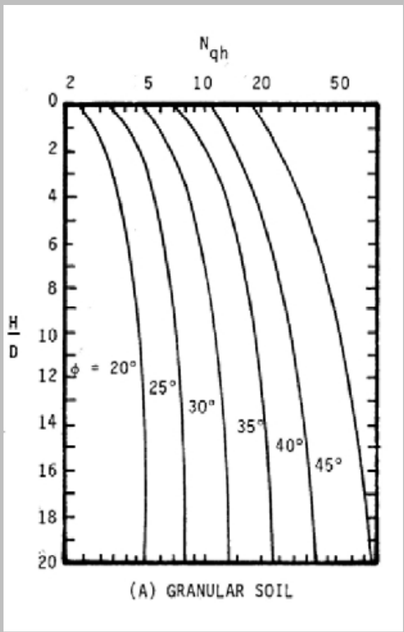


Transverse horizontal soil springs

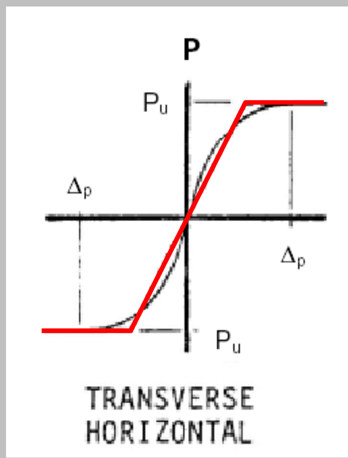
maximum lateral soil force per unit length

$$P_u = N_{qh} \overline{\gamma} HD$$

[passive earth pressure on a horizontally moving shallow footing]



bearing capacity factor for sand (Hanshen, 1961)



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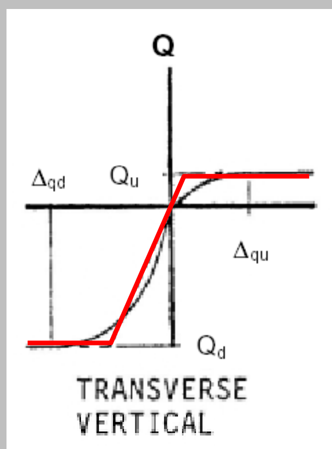
Transverse vertical soil springs

Downward Motions: **the pipeline is assumed to act as a cylindrically-shaped strip footing and the ultimate soil resistance q_u is given by conventional bearing capacity theory. For cohesionless soils the force per unit length is:**

$$q_u = \gamma \cdot H \cdot N_q \cdot D + 0.5 \cdot \gamma \cdot D^2 \cdot N_\gamma$$

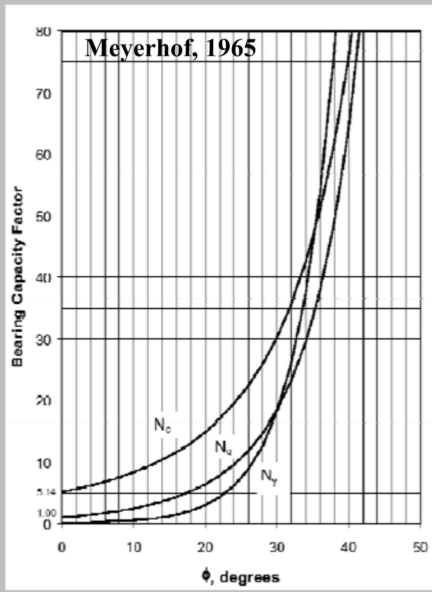


N_q, N_γ = bearing capacity factors for horizontal strip footings, vertically loaded in the downward direction

γ = effective unit weight of soil

Transverse vertical soil springs

Downward Motions: **the pipeline is assumed to act as a cylindrically-shaped strip footing and the ultimate soil resistance q_u is given by conventional bearing capacity theory. For cohesionless soils the force per unit length is:**



$$q_u = \gamma \cdot H \cdot N_q \cdot D + 0.5 \cdot \gamma \cdot D^2 \cdot N_\gamma$$

N_q, N_γ = bearing capacity factors for horizontal strip footings, vertically loaded in the downward direction

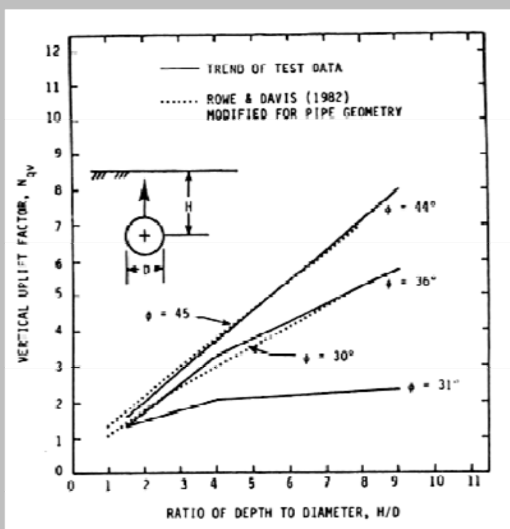
γ = effective unit weight of soil

Transverse vertical soil springs

Upward motions: **Based on tests performed with pipes buried in dry uniform sand, the relationship between the force q and the vertical upward displacement z, has been shown to vary according to the following hyperbolic relation**

$$q = \frac{z}{A + B \cdot z}$$

$A = 0.07\ z_u / q_u$
$B = 0.93/q_u$
ultimate uplift resistance $q_u = \gamma \cdot H \cdot N_{qv} \cdot D$

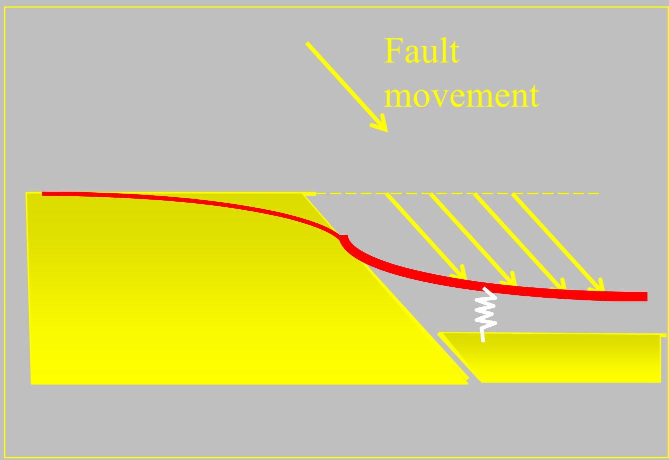


Trautmann & O' Rourke, 1983

The vertical uplift factor N_{qv} is a function of the depth to diameter ratio H/D and the friction angle of the soil φ

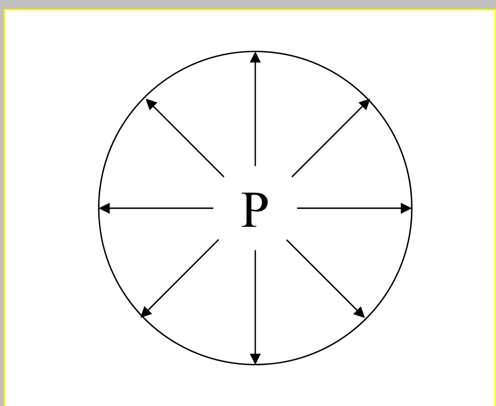
Load modeling

Fault activation is modeled as an imposed displacement
at the base of the soil springs attached on one side of the fault line



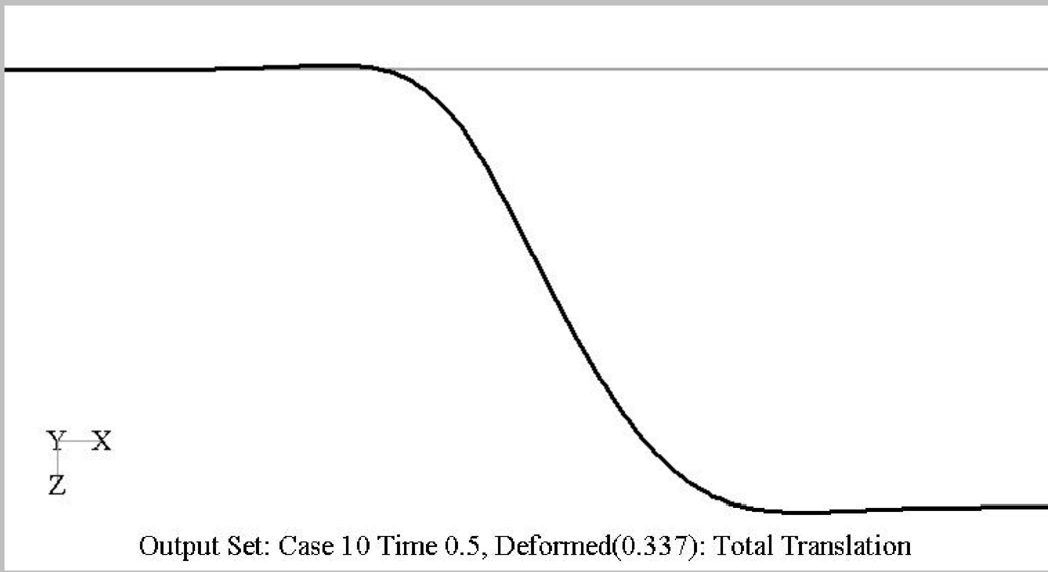
Load modeling

For the Mixed Model the internal pressure is modeled as a uniform load normal to the internal face of the shell elements. The nominal pressure is 10.2 MPa according to the specification of ASME.



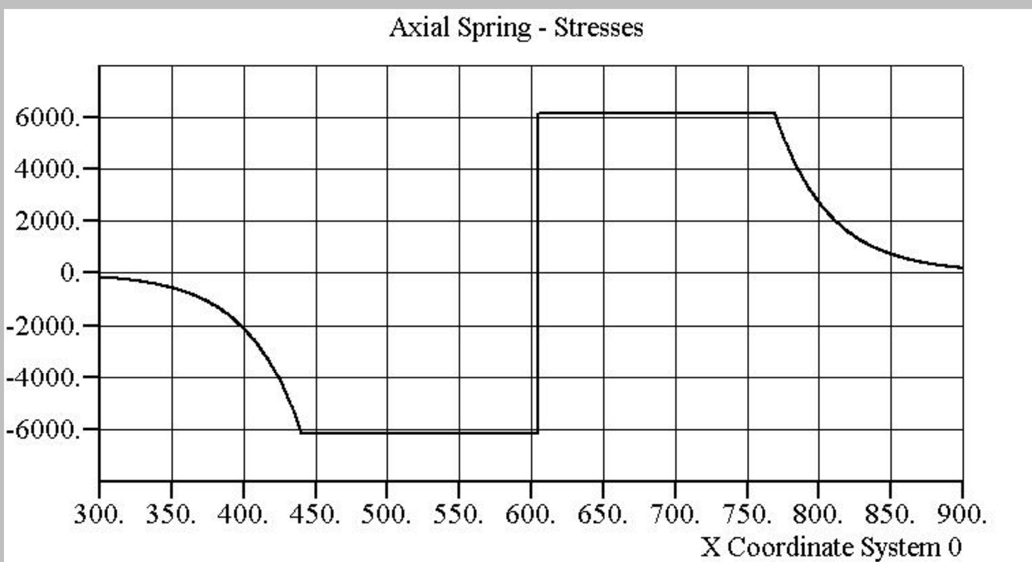
The displacement field is imposed in a number of steps to capture eventual non-linearities in the pipe's response with regard to fault rapture

Normal fault results



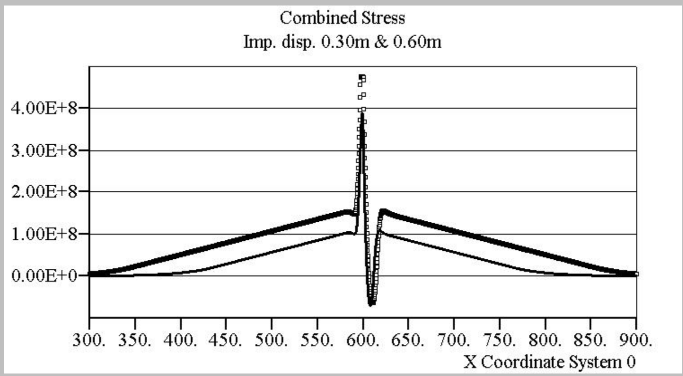
Beam Model – Deflected shape

Normal fault results

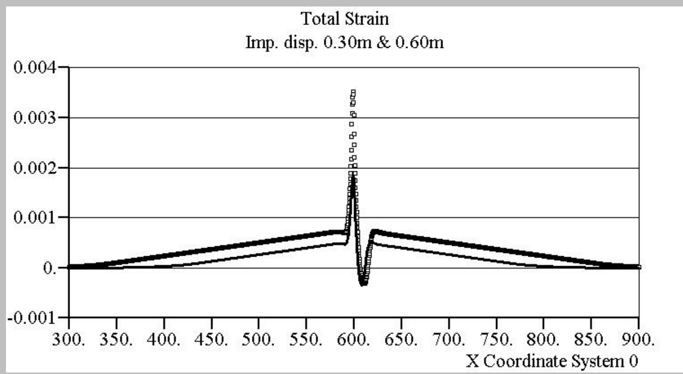


Beam Model – Axial spring stresses

Normal fault results

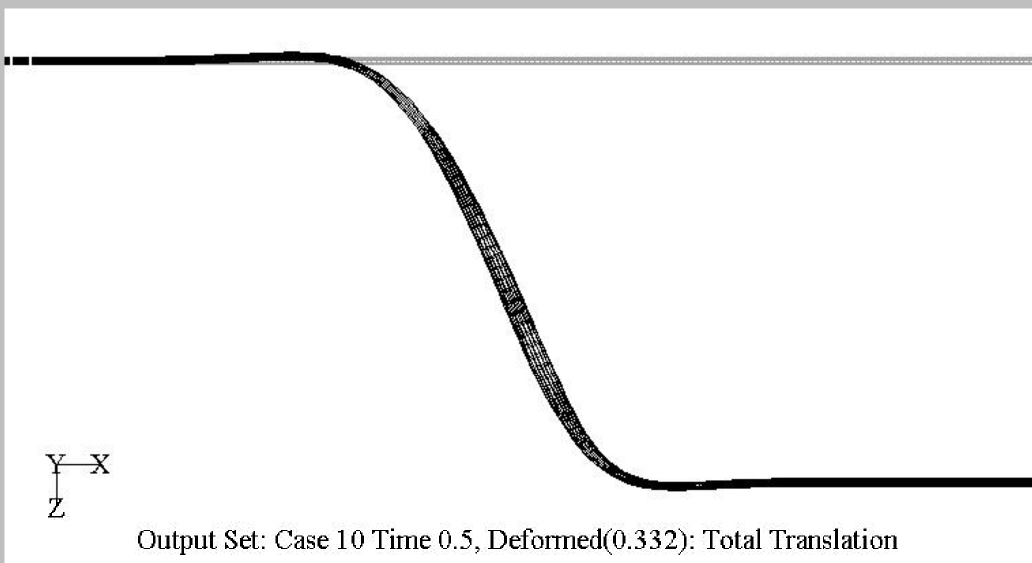


Beam Model – Stresses



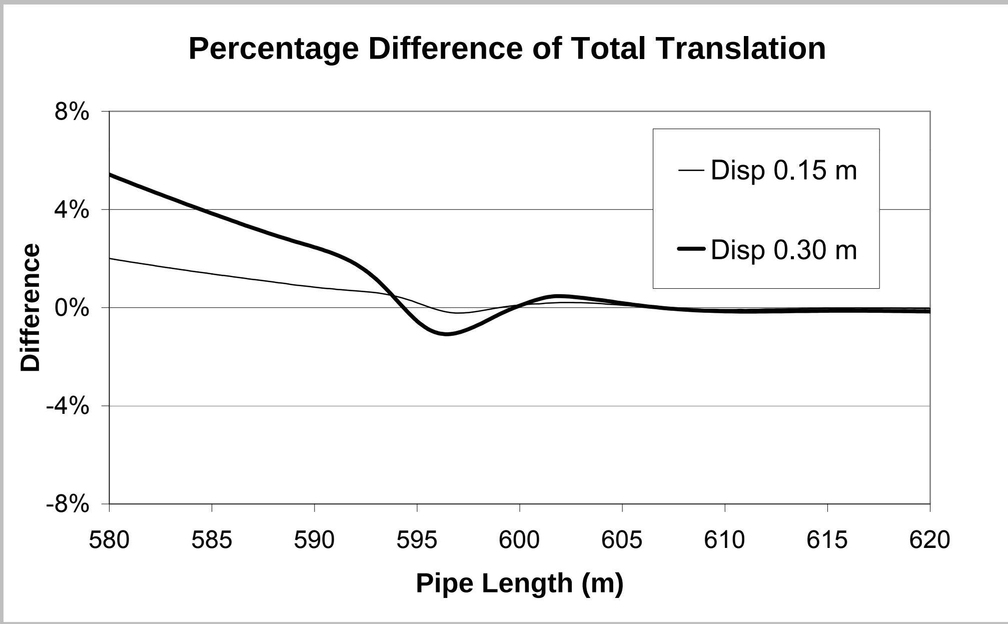
Beam Model – Strains

Normal fault results



Mixed Model – Deflected shape

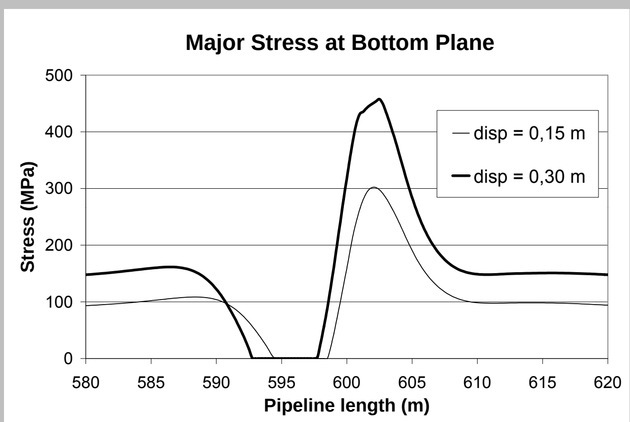
Normal fault results



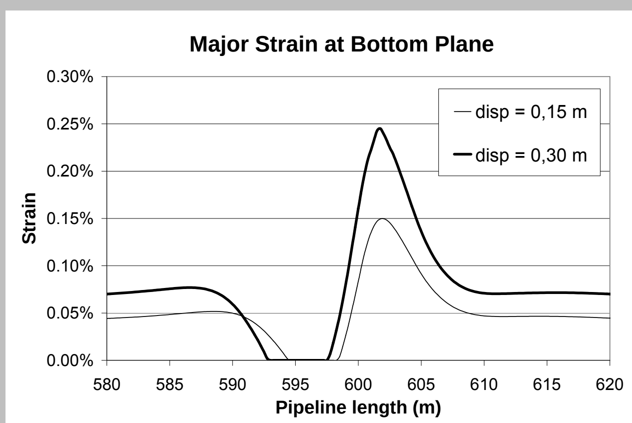
Mixed Model – % difference of total translation due to internal pressure

Normal fault results

No pressure



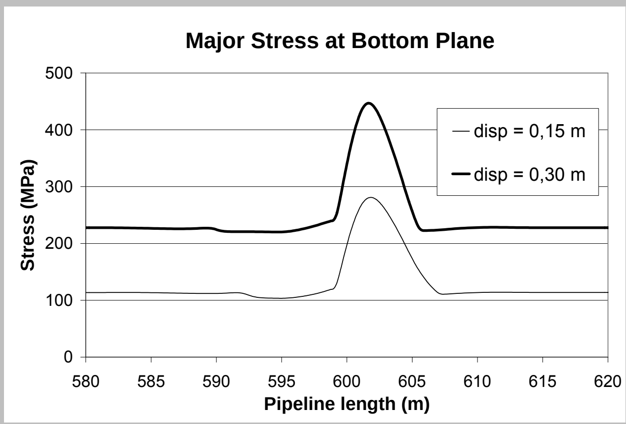
Mixed Model – Stresses



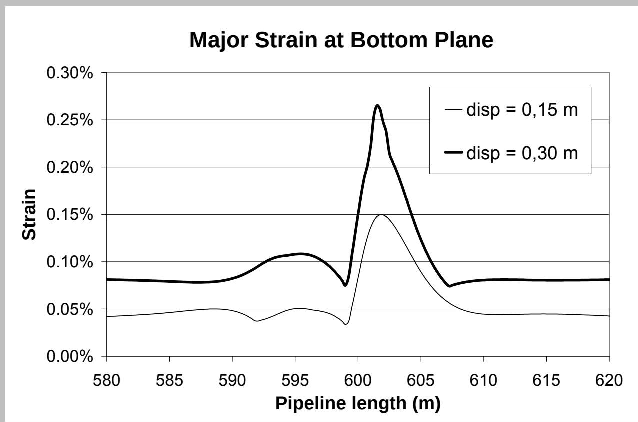
Mixed Model – Strains

Normal fault results

Pressure 10.2 MPa

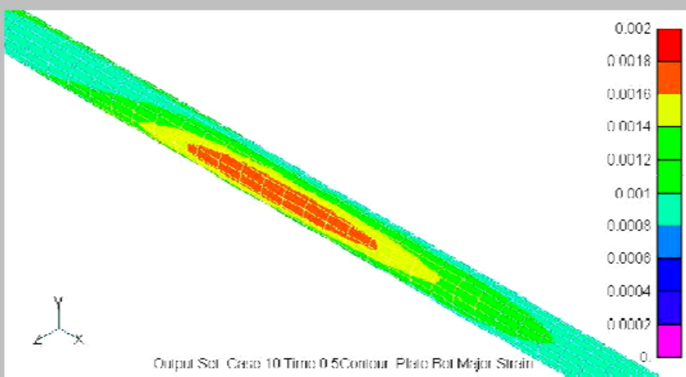


Mixed Model – Stresses



Mixed Model – Strains

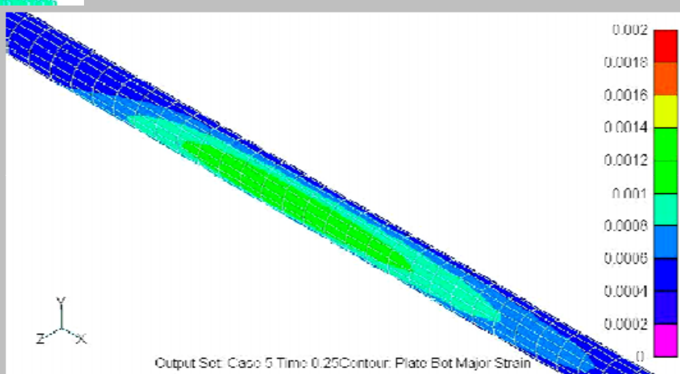
Normal fault results



Mixed Model – Strains

With pressure

Displacement 0.30m

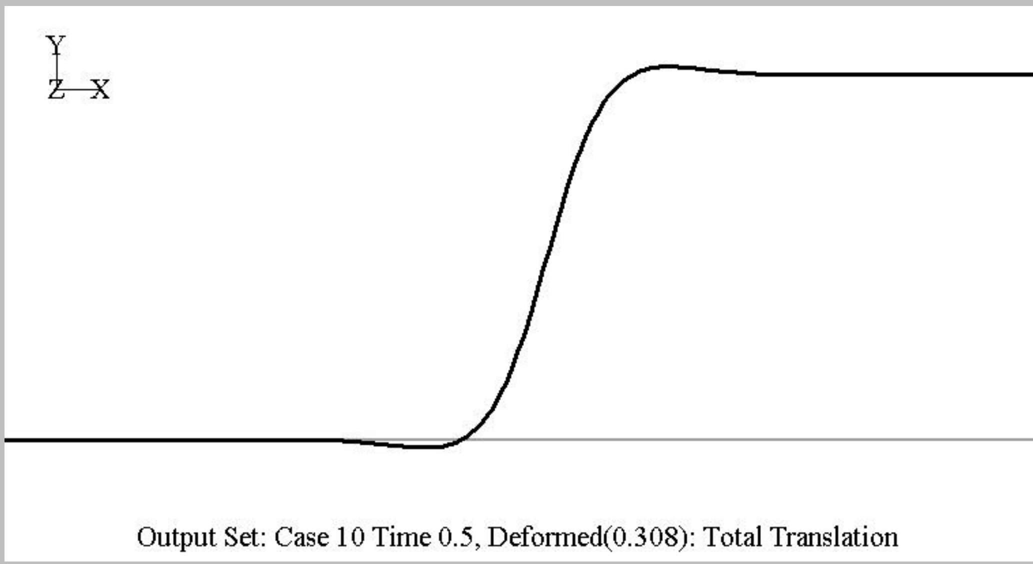


Mixed Model – Strains

With pressure

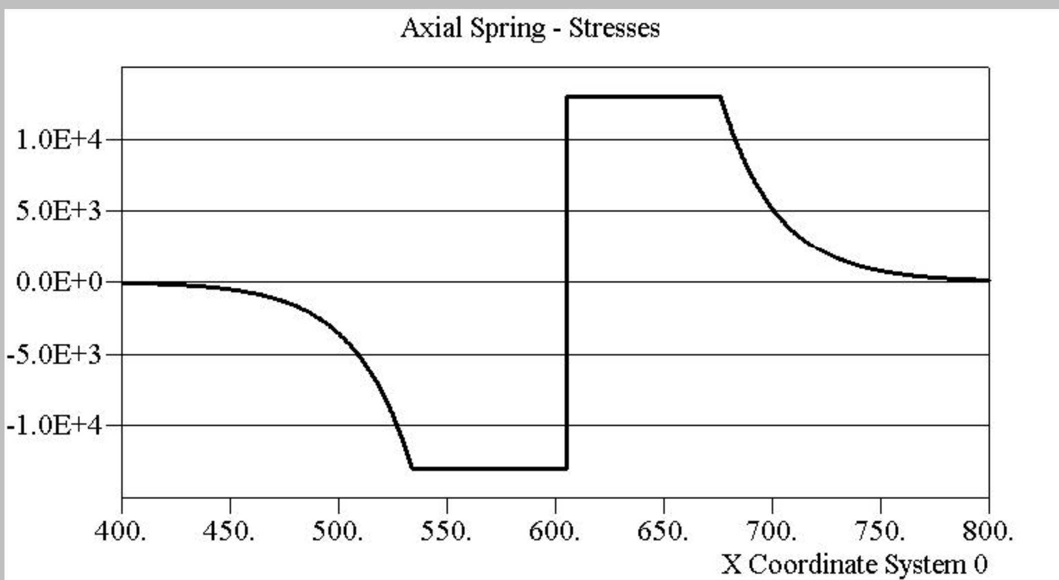
Displacement 0.15m

Strike slip fault results



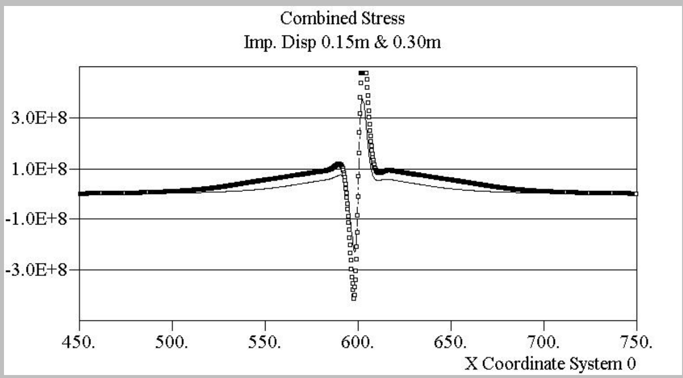
Beam Model – Deflected shape

Strike slip fault results

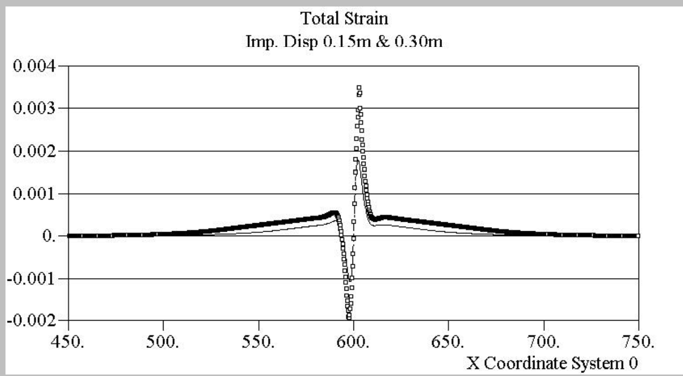


Beam Model – Axial spring stresses

Strike slip fault results

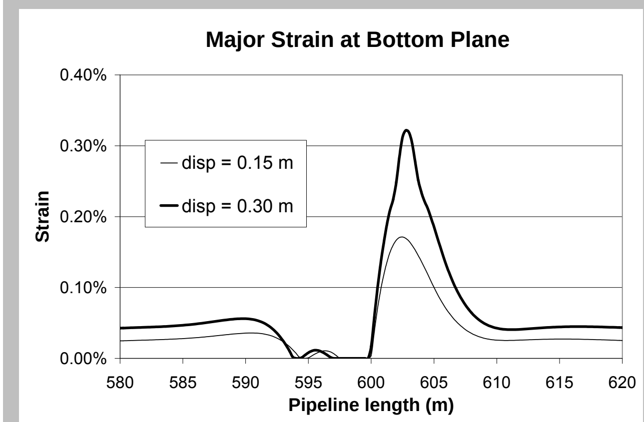


Beam Model – Stresses

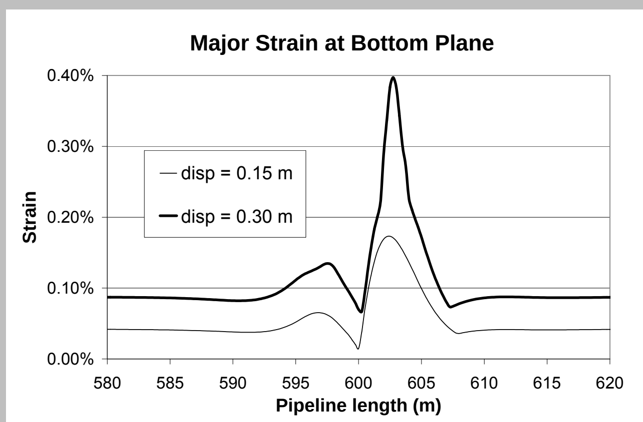


Beam Model – Strains

Strike slip fault results



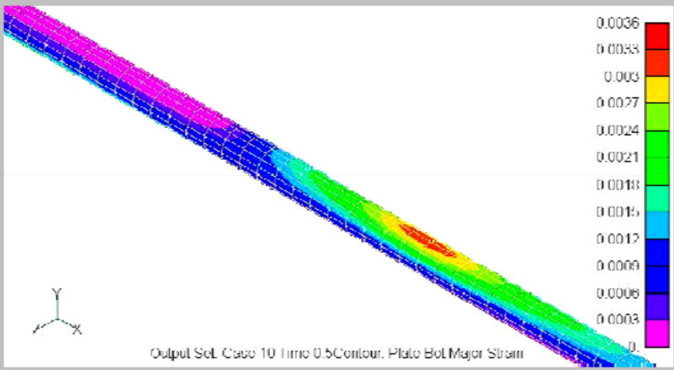
No Pressure
Mixed Model – Stresses



Mixed Model – Strains

Pressure 10.2 MPa

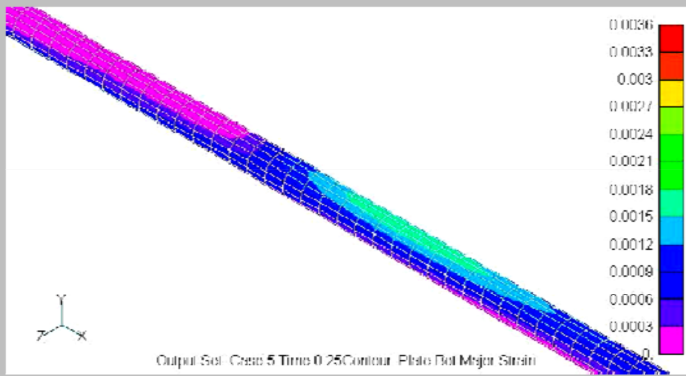
Strike slip fault results



Mixed Model – Strains

With pressure

Displacement 0.30m



Mixed Model – Strains

With pressure

Displacement 0.15m

Analysis conclusions

- No failure of the pipeline at zones of active faults is expected.
- For the "strike-slip" fault case yielding of the pipeline at places near the fault is anticipated.
- Good agreement of the results from the two models for both studies is observed.

Construction countermeasures

<u>*Heavy wall sections near active faults*</u> **This measure will increase the pipeline stiffness relative to that of the backfill and will lead to smaller curvatures and smaller internal strains**

<u>Arrangement of loose backfill around the pipe</u>, extending beyond the anticipated displacement along the critical zone

<u>Wrap the pipeline with a proper, friction reducing geotextile</u> which will provide a lower friction coefficient

<u>Enclose the pipeline within a casing,</u> so that the pipeline can move freely along the intensely distorted length, on both sides of the fault trace

Construction countermeasures