NRC/PG&E Open Meeting, San Francisco CA
Diablo Canyon Independent Spent Fuel Storage Installation

Properties of Subsurface Materials

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April 11, 2002

Purpose

- Characterize subsurface materials for ISFSI, CTF and Transport Route for
  - Foundation properties
  - Slope stability assessments

Subsurface Materials Assessed

- Rock
  - Dolomite
  - Sandstone
  - Friable rock
- Clay Beds

Subsurface Materials Properties - Rock

- Density
- Shear Wave Velocities
- Young's Moduli and Poisson's Ratios
- Shear Strength

Rock Properties - Density

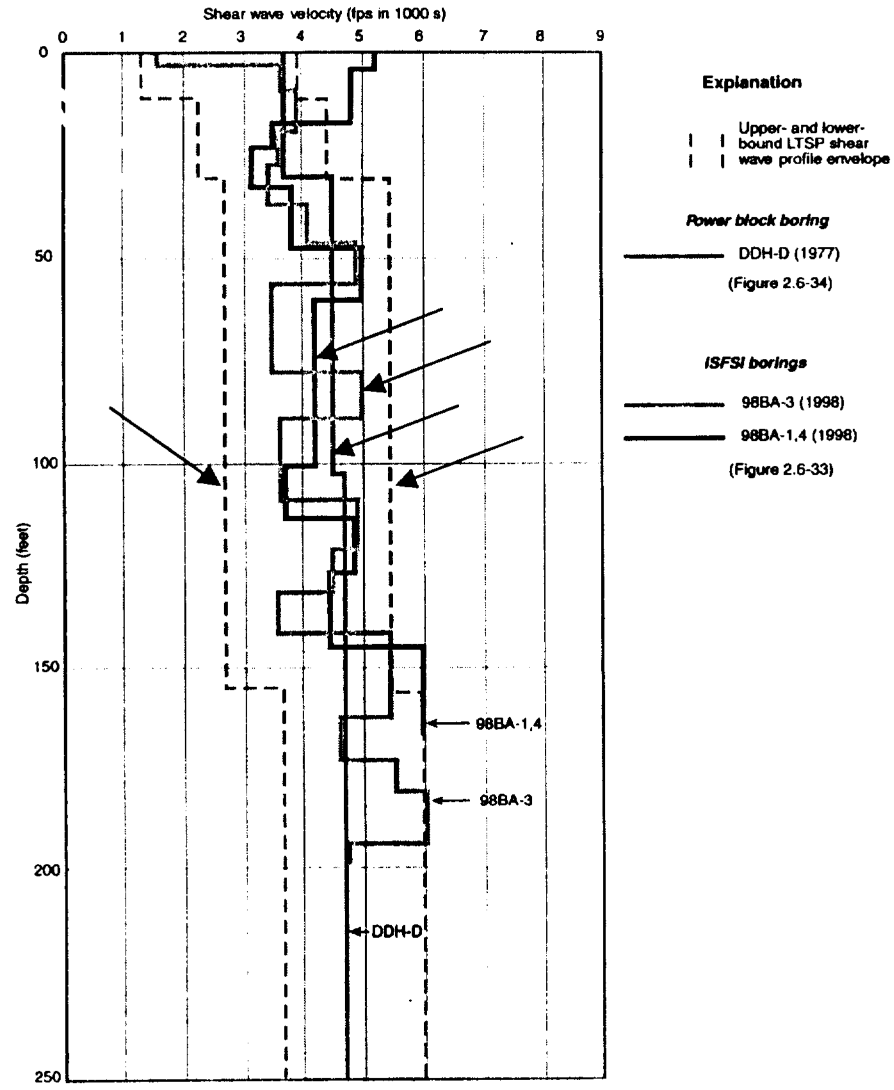
- Determined from lab tests of rock core samples of dolomite and sandstone
- 140 pcf ± 8 pcf for all rock

From SAR Section 2.6.4.3.1; Data Report I, Table 1

Rock Properties - Shear Wave Velocities

- Obtained from suspension logging of borings at ISFSI site
- Compared with velocities obtained in previous investigations at power block

From SAR Section 2.6.5.1.3.2 Figs. 2.6-33, 34, and 35; SAR 2.6.1.10; Data Report C



Rock shear wave velocities

From SAR Figure 2.6-35

Young's Moduli and Poisson's Ratios

- From suspension velocities for rock mass
- Compared with lab tests of rock core samples of dolomite and sandstone

From SAR Section 2.6.4.3.1; Data Report I

Results - Rock Properties

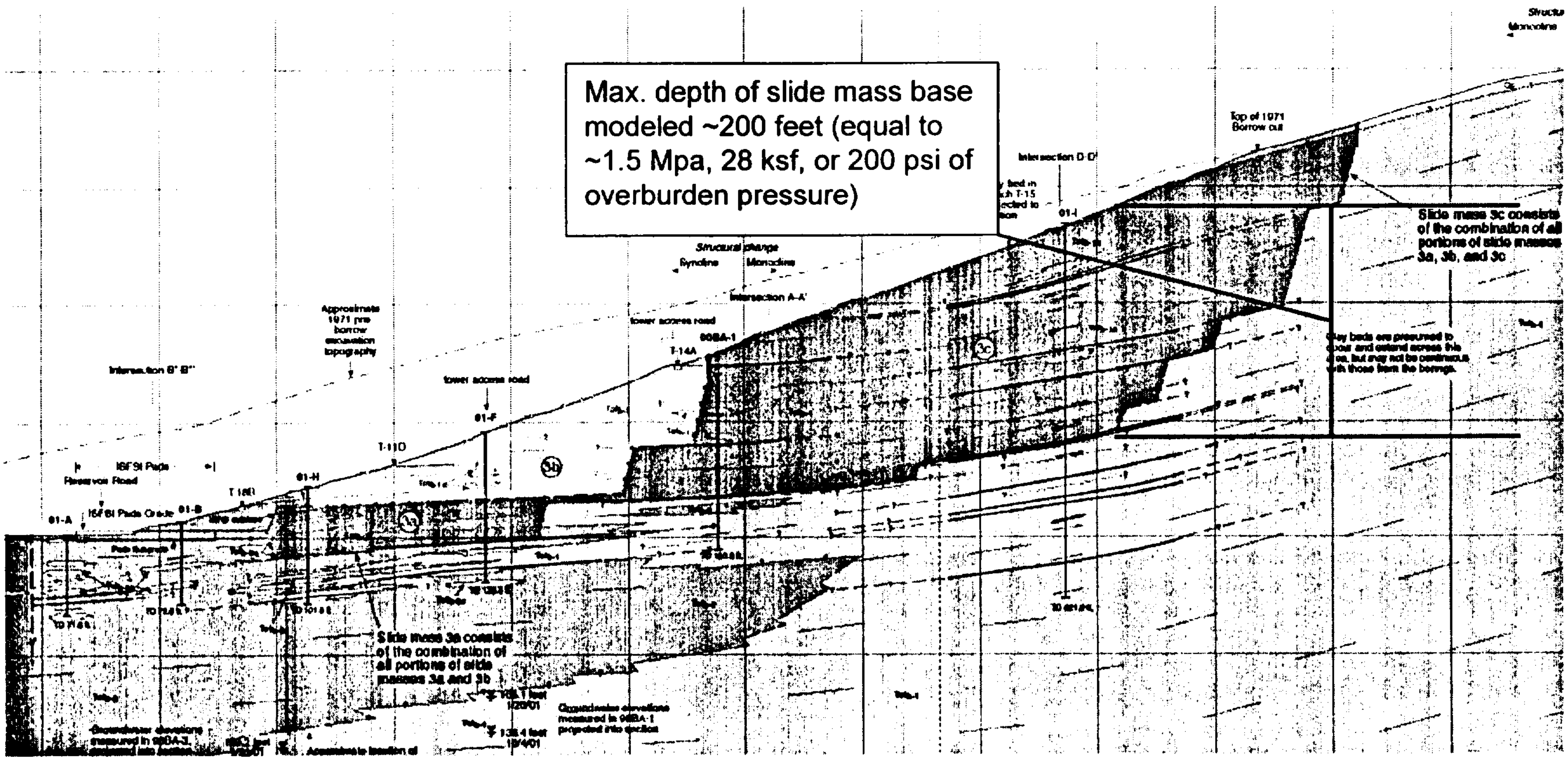
- Young's moduli
  - 1.3×10^6 to 1.5×10^6 psi, non-friable rock
  - 0.20×10^6 to 0.21×10^6 psi, friable rock
- Poisson's ratios
  - 0.22 to 0.37, non-friable rock
  - 0.23 to 0.31, friable rock

From SAR Section 2.6.4.3.1; Data Report I

Rock Strength Parameters depend on:

- Type of rock
  - Dolomite, sandstone
  - Friable rock

- Scale of rock mass analyzed
  - Large scale
    - Rock slide mass
  - Small scale
    - Rock wedges



Potential large scale slide mass

From SAR Figure 2.6-49

Large-scale Slide Mass

- Strength controlled by rock mass
  - Discontinuities (joints, bedding planes, faults)
  - Size of intact blocks

- Strength of rock mass based on Hoek-Brown method

From SAR Section 2.6.5.1.2.3

NRC Request for Clarification:

- *"Discuss the technical basis (data and analysis) to justify the rock-mass friction angle of 50 degrees used to characterize the rock-mass strength of dolomite and sandstone."*

Hoek-Brown Input Parameters for Sandstone and Dolomite

- Geologic Strength Index (GSI) values as a function of discontinuity condition and spacing

- Material index (m_i) values as a function of rock type and texture

- Unconfined compressive strength (σ_{ci}) of intact rock samples

Distribution of Hoek-Brown Input Parameter Values

Dolomite	mean plus one sigma	mean	mean minus one sigma
GSI	65	56	46
m_i	17	15	13
σ_{ci}	47	32	18

Sandstone	mean plus one sigma	mean	mean minus one sigma
GSI	68	65	62
M_i	19	18	17
σ_{ci}	31	22	12

From Calculation GEO.DCPP.01.19

Example Hoek Brown strength envelope

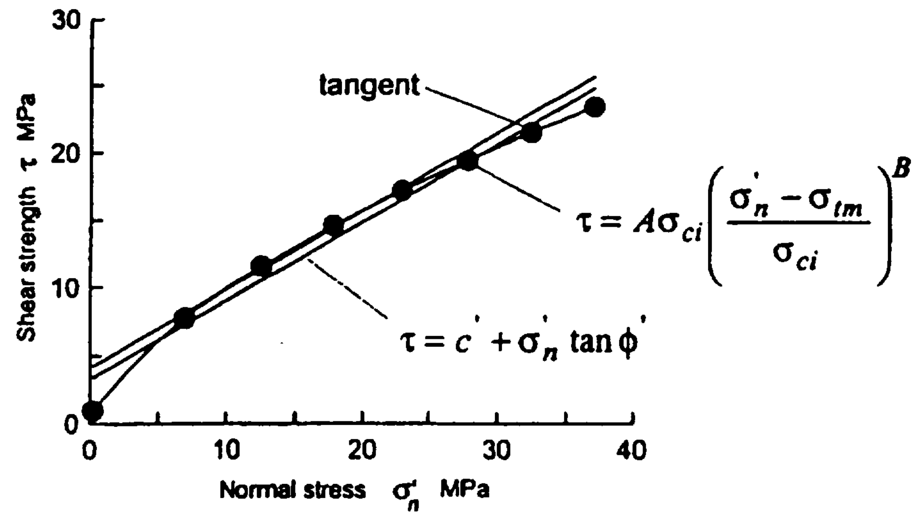
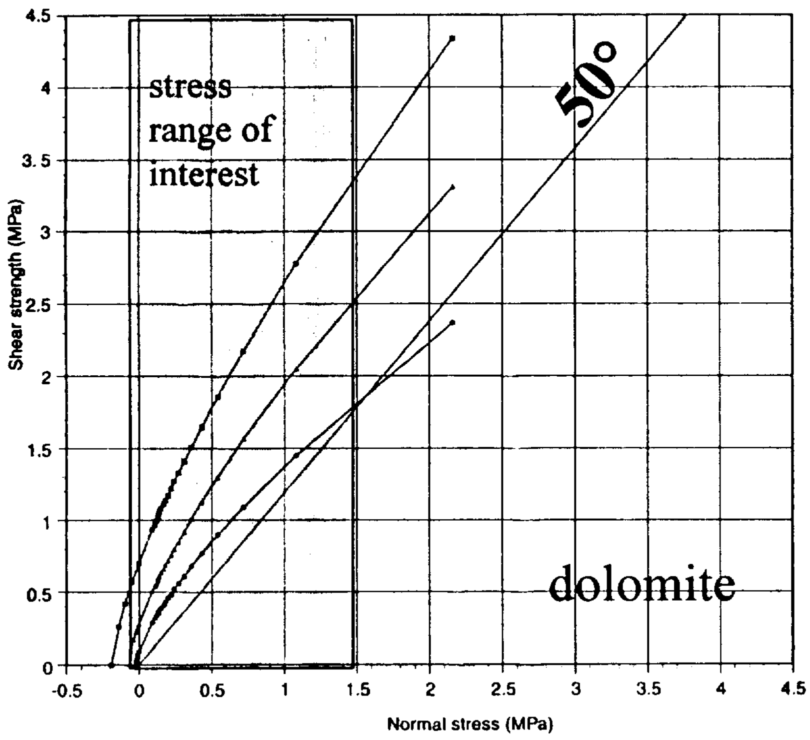
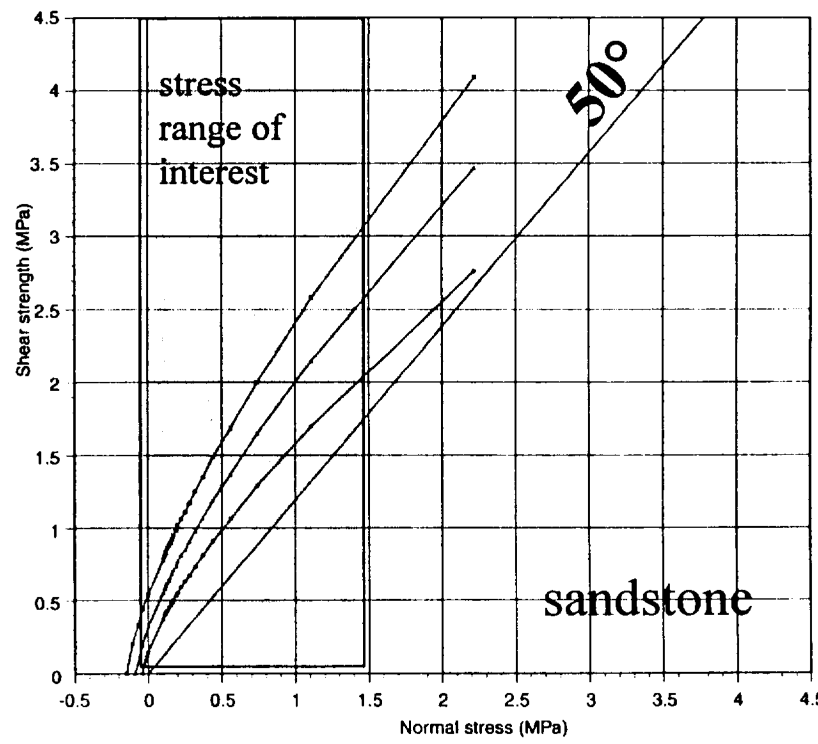


Figure 11.8: Plot of results from simulated full scale triaxial tests on a rock mass defined by a uniaxial compressive strength σ_{ci} = 85 MPa, a Hoek -Brown constant m_i = 10 and a Geological Strength Index GSI = 45.

Rock Mass Strength envelopes (Hoek-Brown)



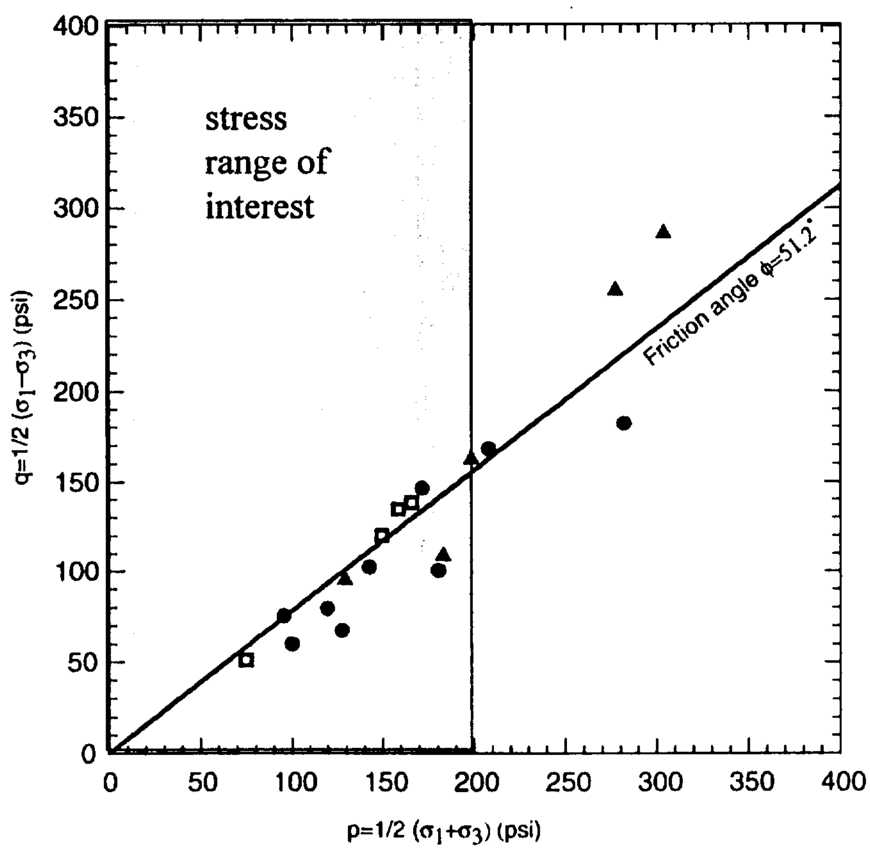


From SAR Figures 2.6-53 and 2.6-54; Calculation GEO.DCPP.01.19

Properties of Friable Sandstone and Dolomite

- Strength not scale dependent since relatively homogeneous (discontinuities have weathered to consistency of rock fabric)
- Samples tested in the lab measured total and effective stresses

Friable Rock Total Stress Strength envelope

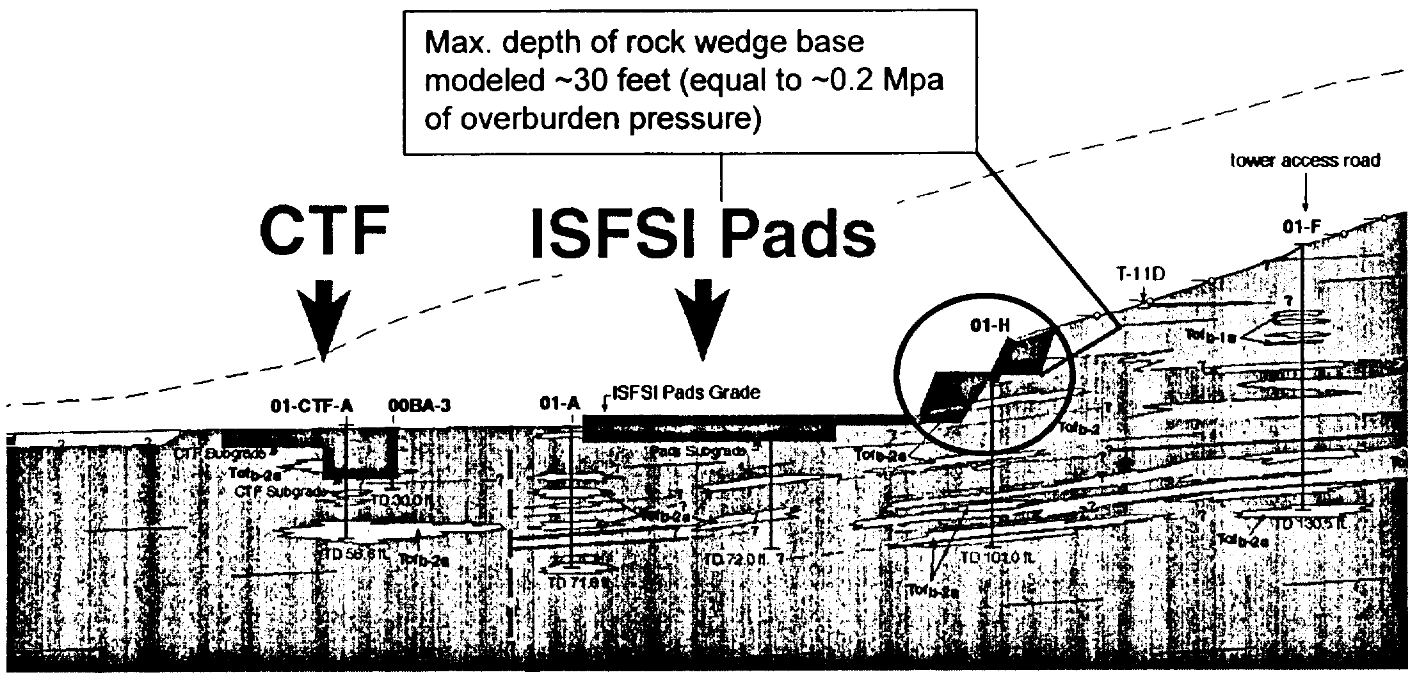


From SAR Figure 2.6-55

Small-Scale Rock Wedge

- Strength controlled entirely by discontinuities

- Strength of discontinuities based on Barton-Choubey method



Potential rock wedges

From SAR Figure 2.6-47

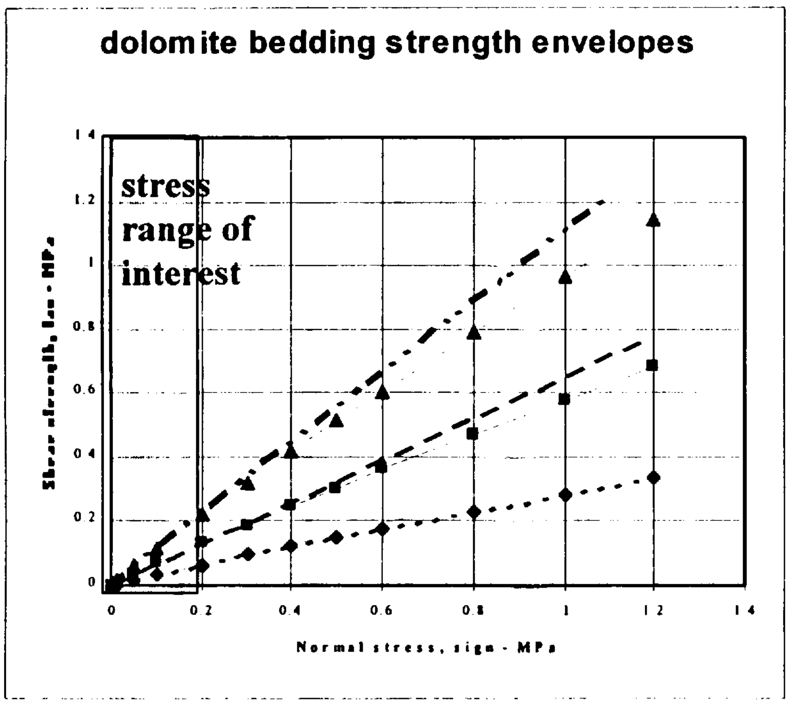
Barton-Choubey Input Parameters for Sandstone and Dolomite

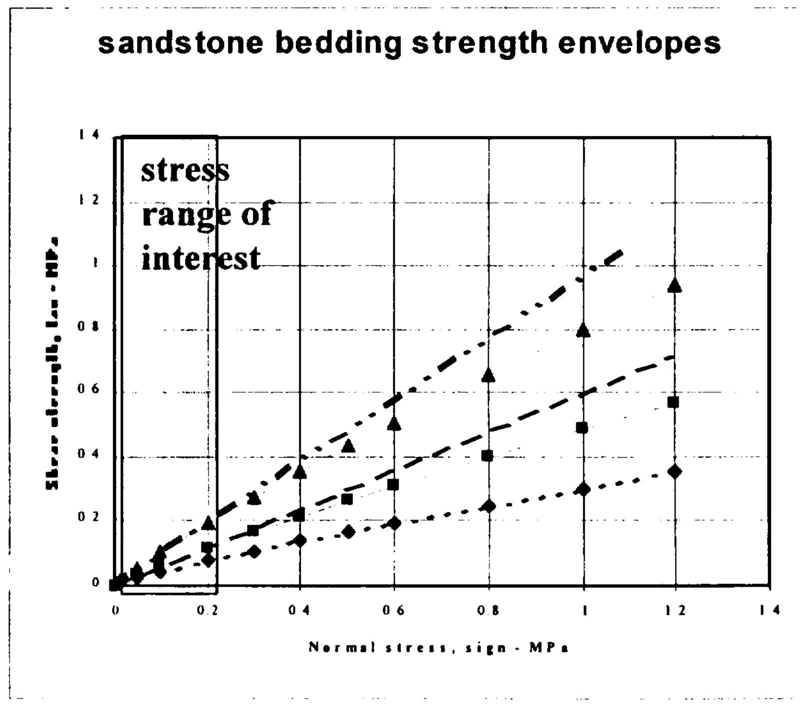
- Base friction angle (ϕ_b) based on lab tests of shear strength of discontinuities
- Joint compressive strength (JCS) based on lab tests of unconfined compressive strength
- Joint roughness coefficient (JRC) values based on field measurements of joints in trenches

From SAR Section 2.6.5.1.2.3)

Straight line fits:
ϕ = 18, 33, & 48°

Straight line fits:
ϕ = 21, 31, & 44°





(shaded zone equals stress range of interest)

From Calculation GEO.DCPP.01.20

Clay Bed Properties

- Density
- Atterberg limits
- Over consolidation ratio
- Shear strength

Density of Clay

- From lab tests of clay samples

- 120 pcf ± 5 pcf

From SAR Section 2.6.4.3.1; Data Report G, Table G-1

Atterberg Limits

- From lab tests of clay samples
- Representative values of Plasticity Index (PI) are between 20 to 40

From Data Report G, Table G-1

Overconsolidation Ratio (OCR)

- Estimated at several points along three clay beds from knowledge of previous ground surface

- Representative values
  - 2 to 5

From GEO.DCPP.01.31

Clay Strength

- Strength of clay beds from lab tests on samples from tower road cut
- Lab results correlated with published PI and OCR relationships
- Post peak and/or large strain strengths used

NRC request for clarification:

- *"Discuss the saturated undrained shear strength of the clay-bed soil."*

Unconsolidated Undrained Shear Strength

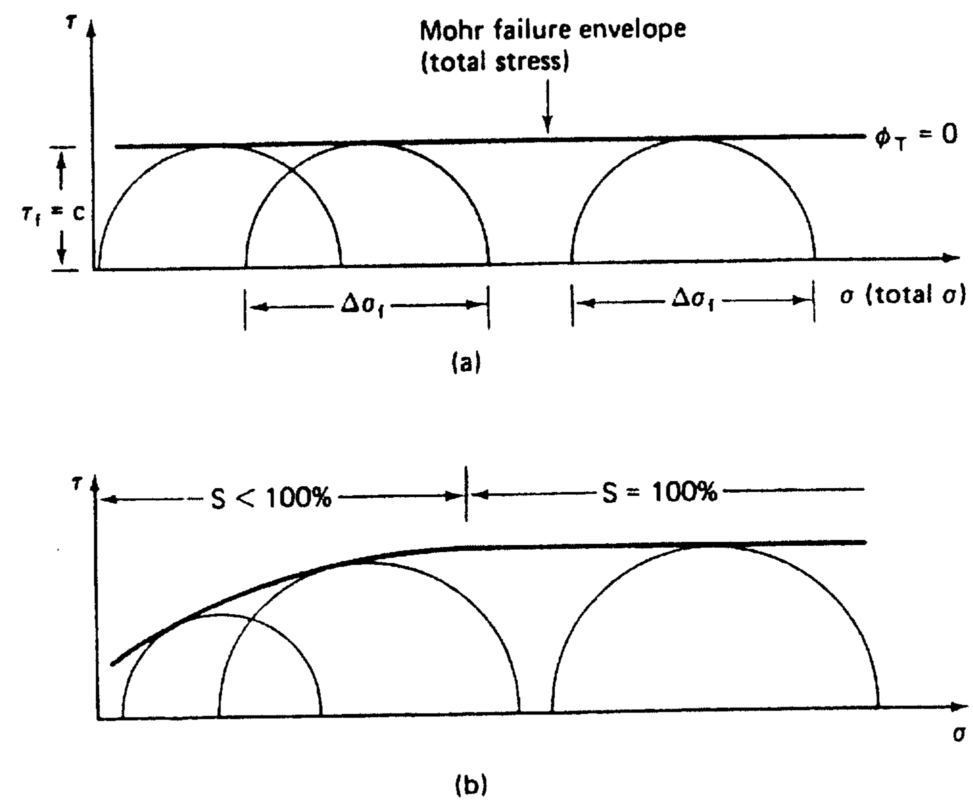
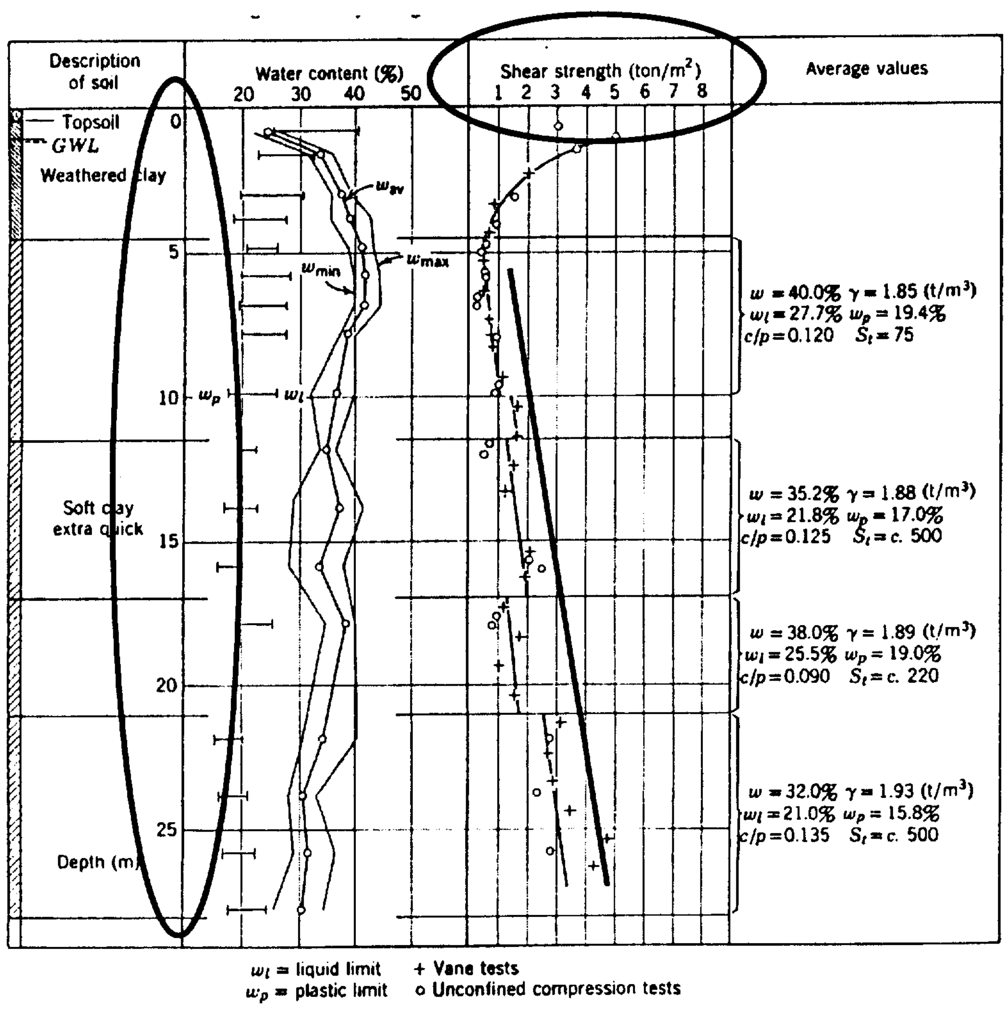


Fig. 11.40 Mohr failure envelopes for UU tests: (a) 100% saturated clay; (b) partially saturated clay.

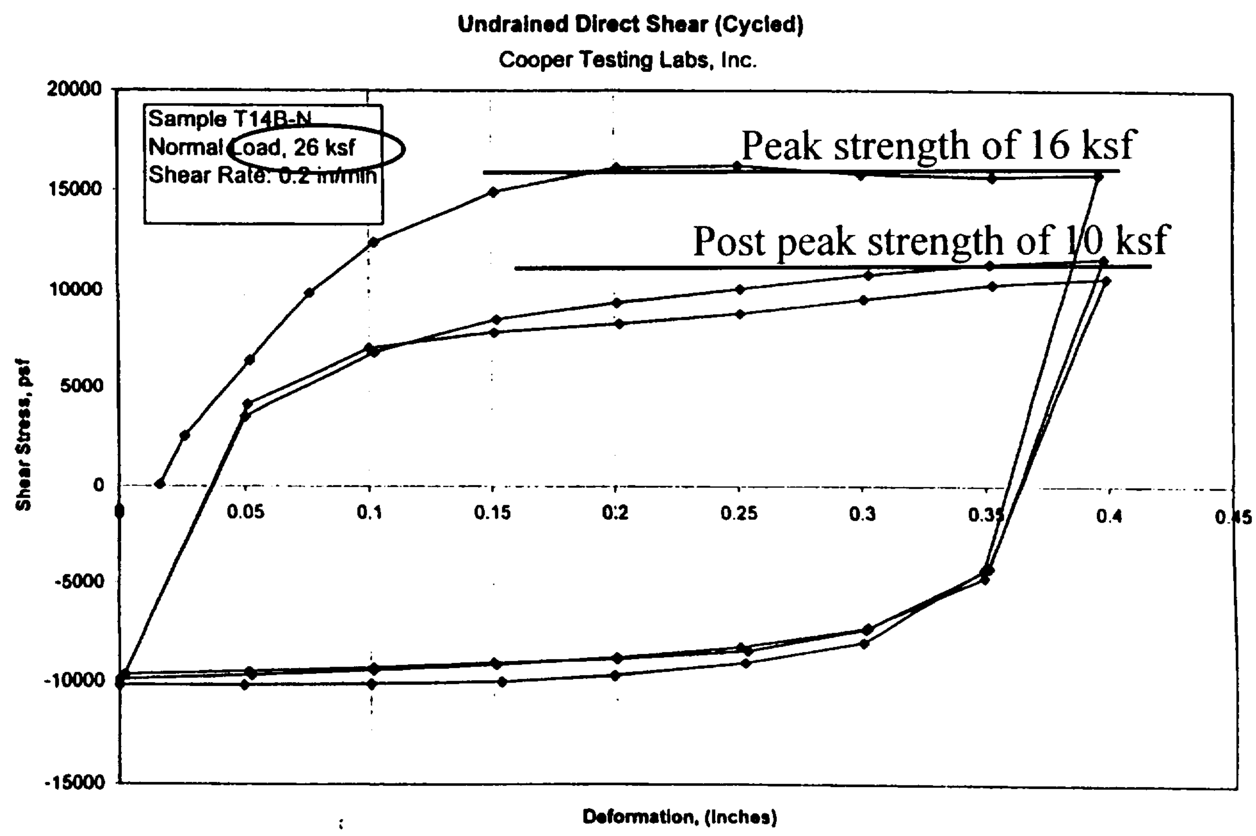
From Holtz and Kovacs, 1981

Consolidated Undrained Shear Strength

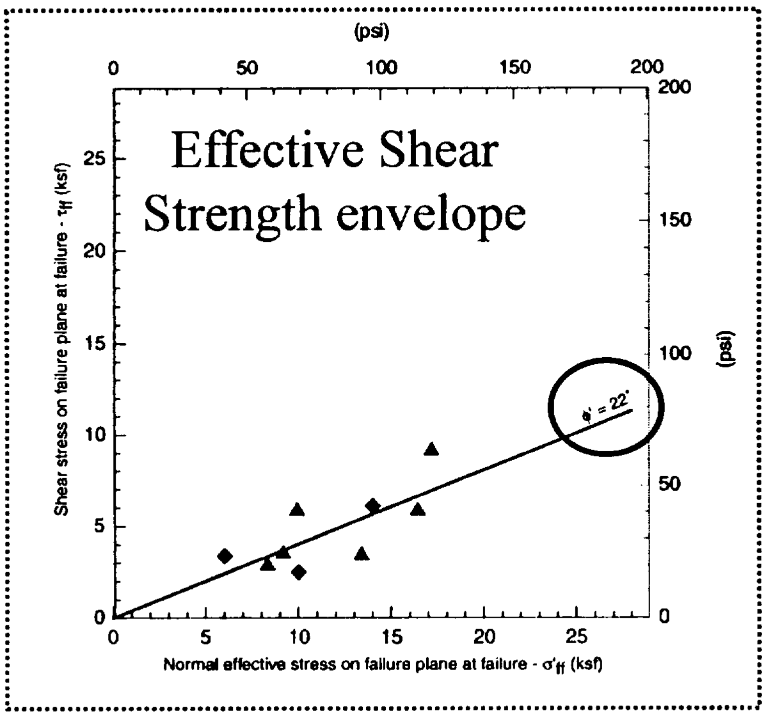


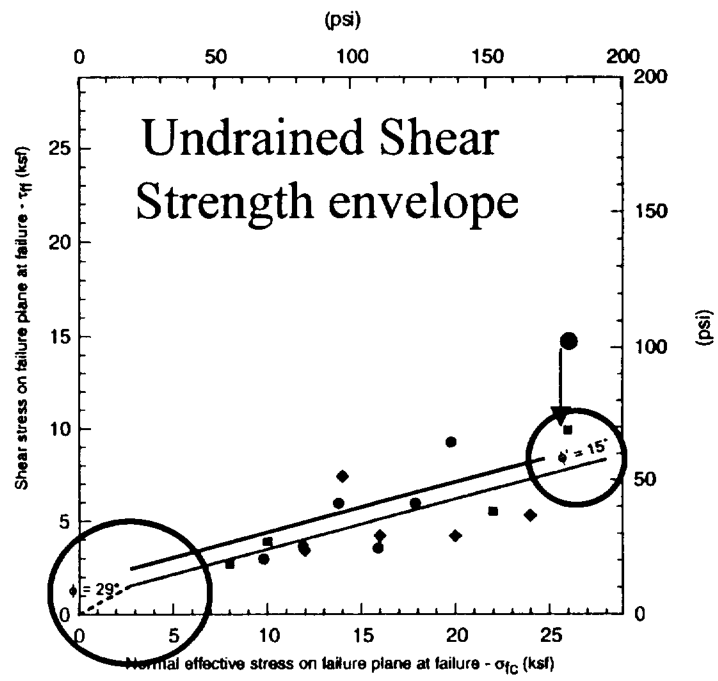
From: Lambe and Whitman, 1969

Laboratory Strength Test Data



Shear Strengths of Clay Bed





From SAR Figure 2.6-50 and 51; Calculation GEO.DCPP.01.31

Summary of Rock Properties

- Density: 140 pcf ± 8 pcf for all rock
- Shear Wave Velocities: > 4000 fps
- Young's Moduli: related to shear wave velocities
- Poisson's ratio: related to shear wave velocities
- Shear Strength:
  - ϕ = 50 degrees for slide masses
  - ϕ = 18 to 31 degrees for rock wedges

Summary of Clay Bed Properties

- Density: 120 pcf ± 5 pcf average
- Atterberg Limits: PI of 20 to 40
- Over Consolidation Ratio: OCR of 2 to 5
- Shear Strength:
  - Effective shear strength: $\phi = 22$ degrees
  - Undrained shear strength:
    - $\phi = 15$ degrees and $c = 500$ psf, or
    - $\phi = 29$ degrees

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Slope Stability Analysis

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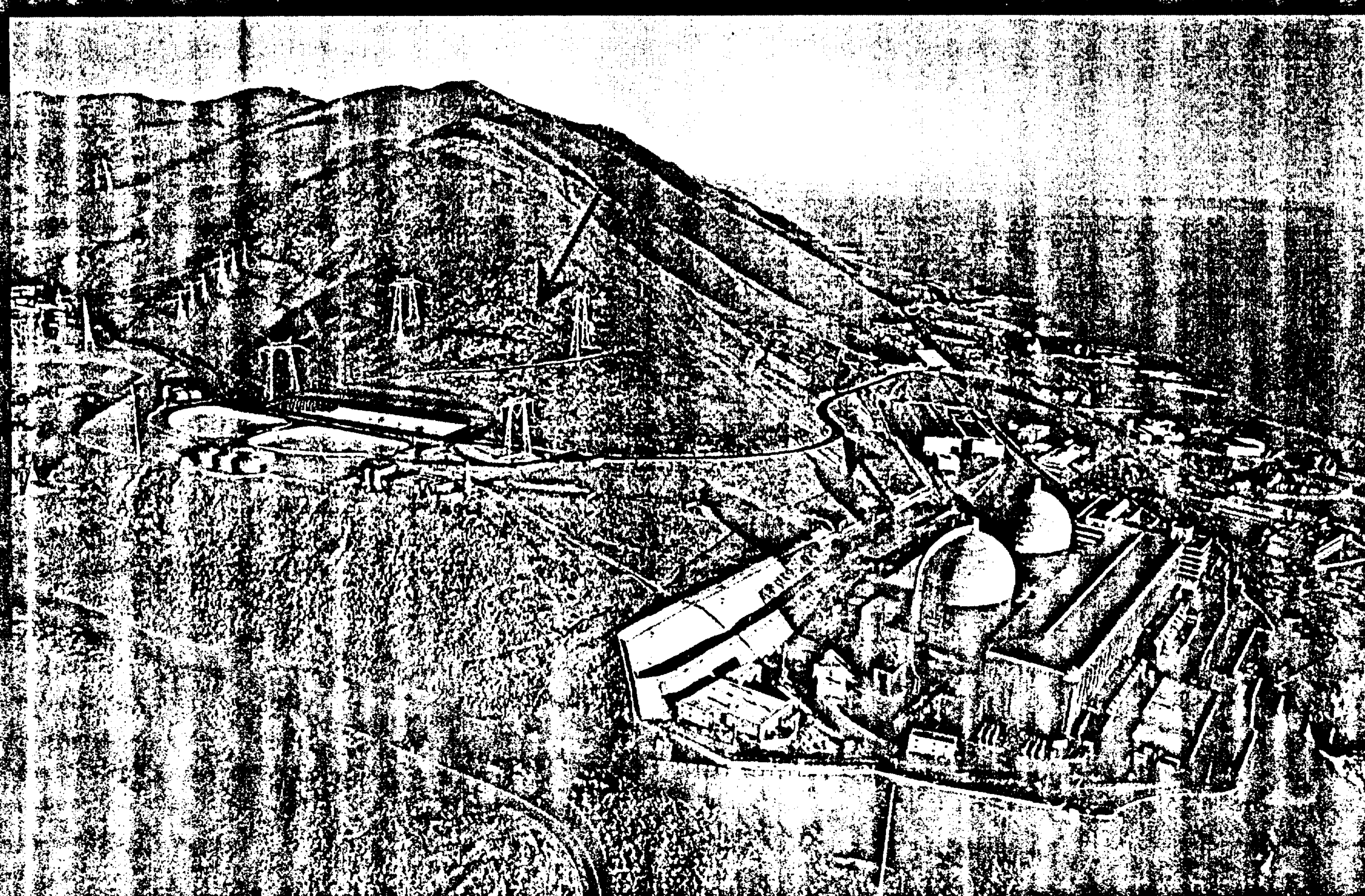
April 11, 2002

Presentation of Slope Stability

■ Hillslope above ISFSI Pads	Joseph Sun
■ Transport route and CTF	Robert White
■ Cutslopes	Jeff Bachhuber

Slope Stability Study Areas



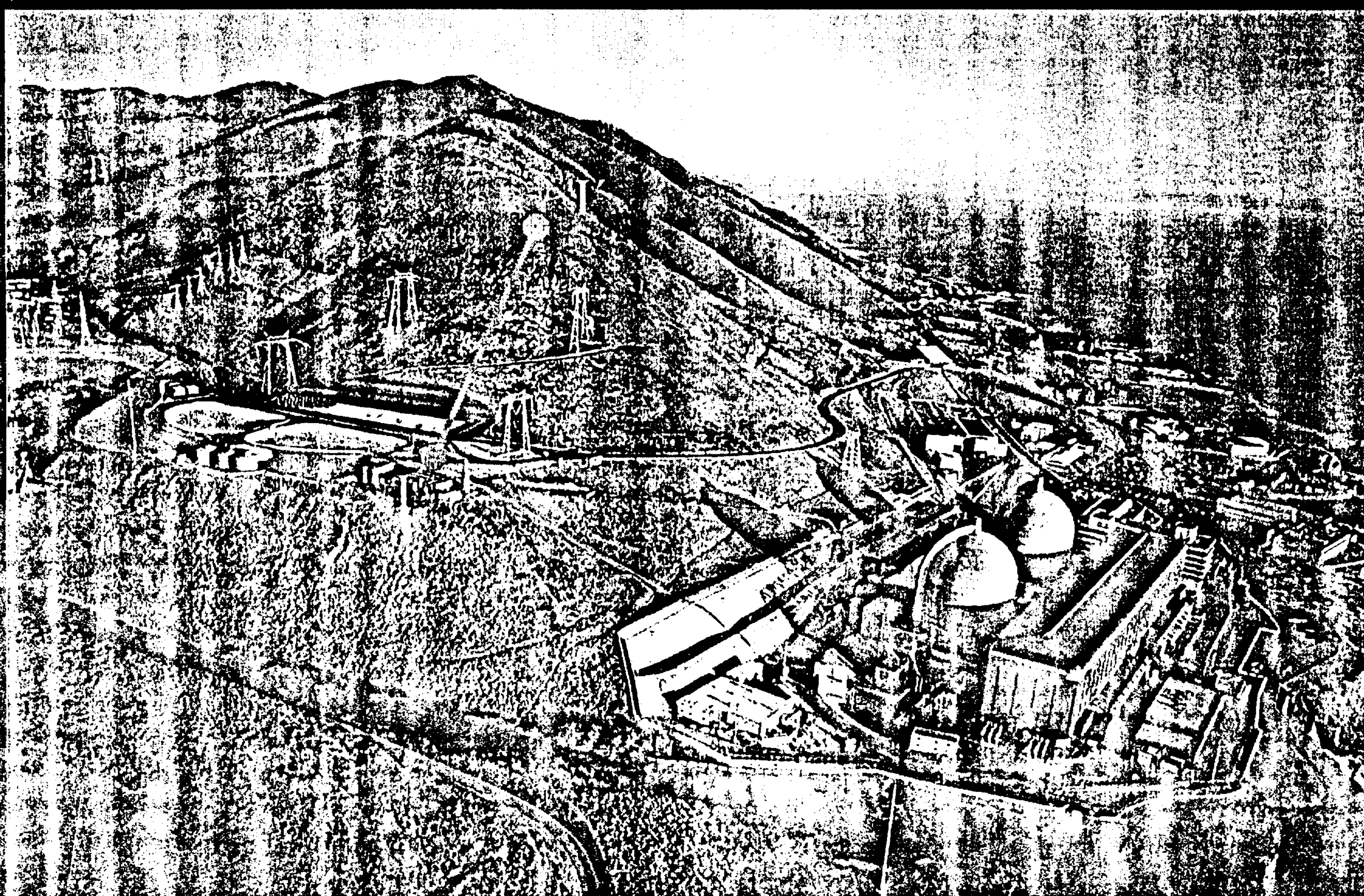
Approach

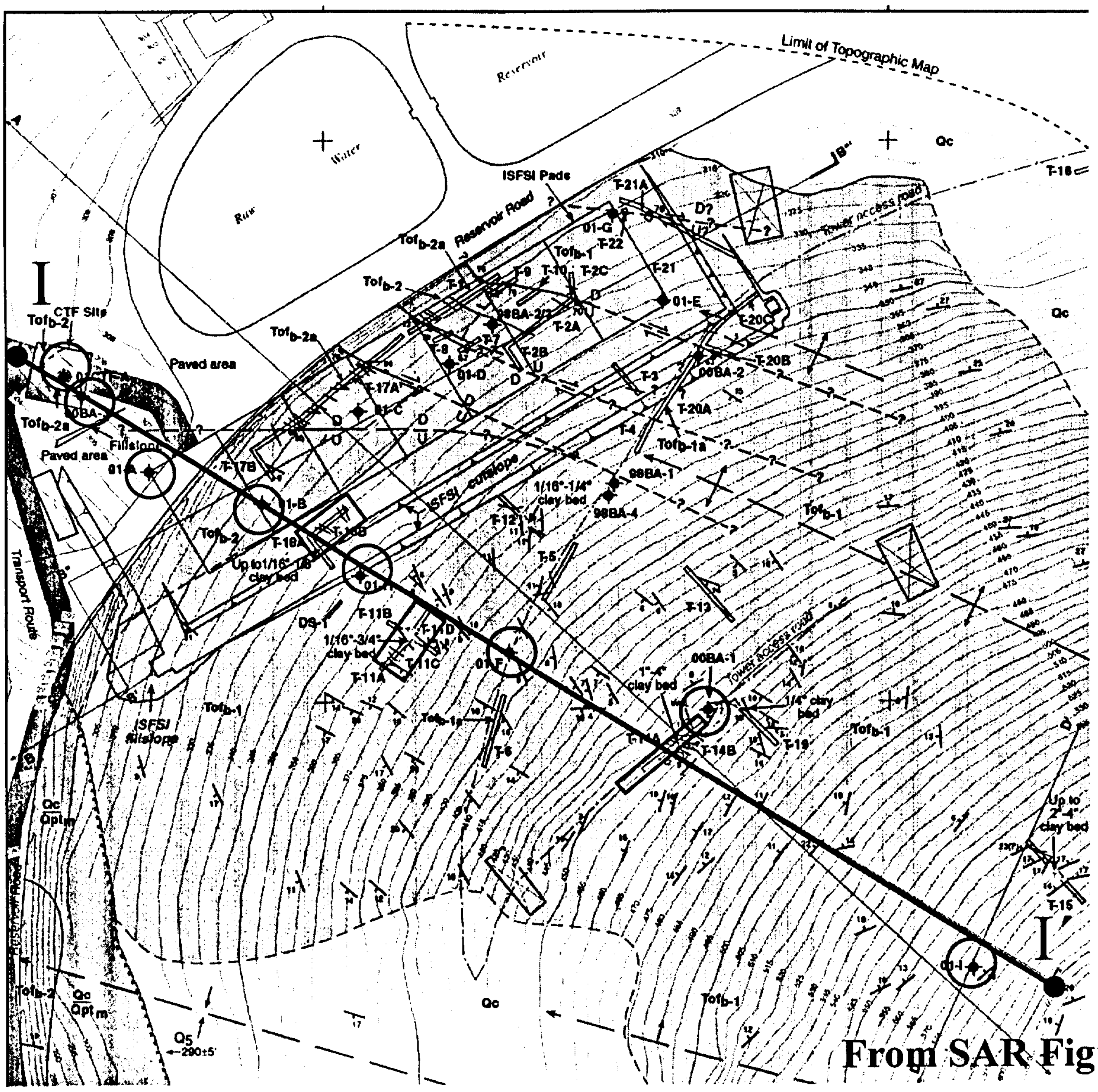
- Select cross section for analyses
- Develop material properties
- Perform slope stability analyses
- Perform dynamic response analyses
- Estimate potential seismic induced displacements of rock masses on clay beds

Approach

- Select cross section for analyses
- Develop material properties
- Perform slope stability analyses
- Perform dynamic response analyses
    - Address NRC request for clarification on vertical ground motions
- Estimate potential seismic induced displacements of rock masses on clay beds

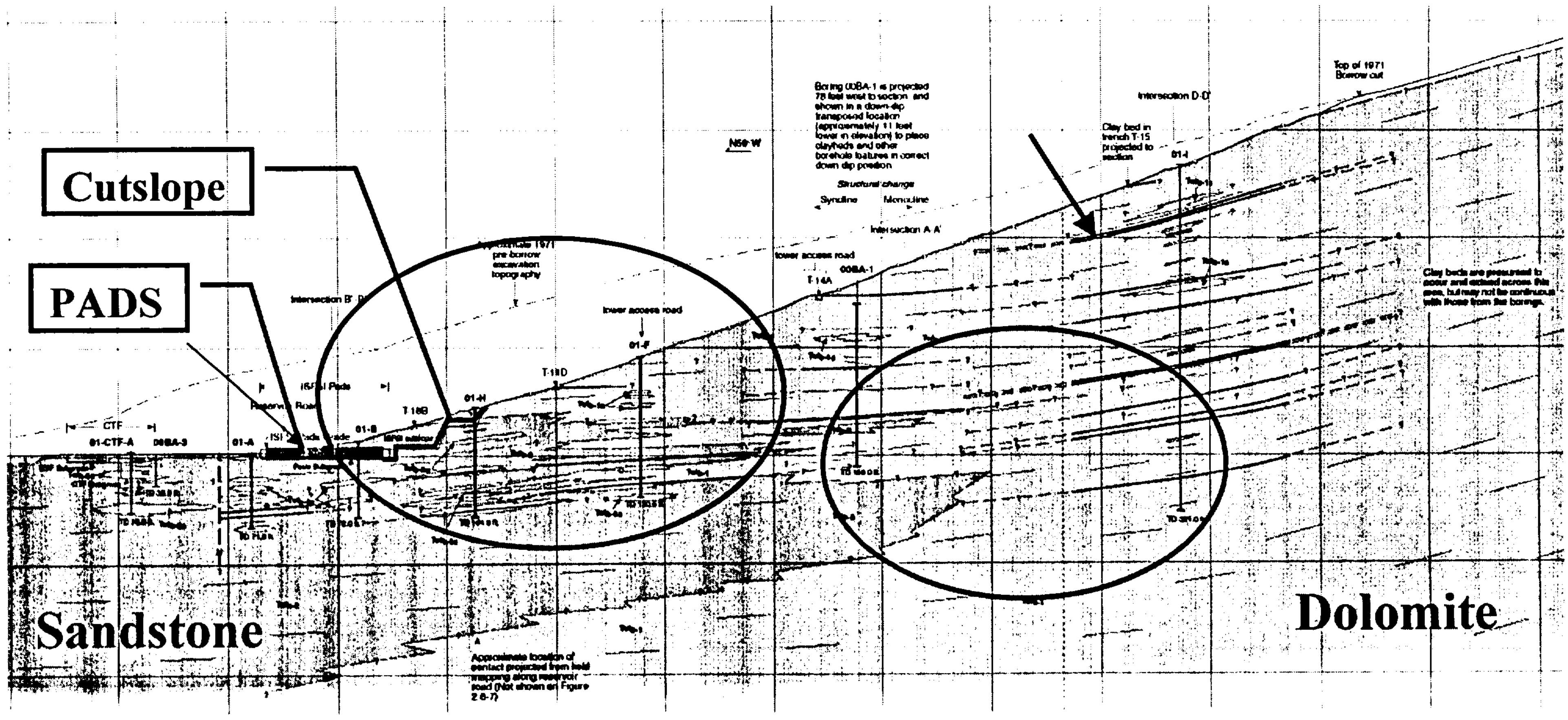
Analysis Cross Section





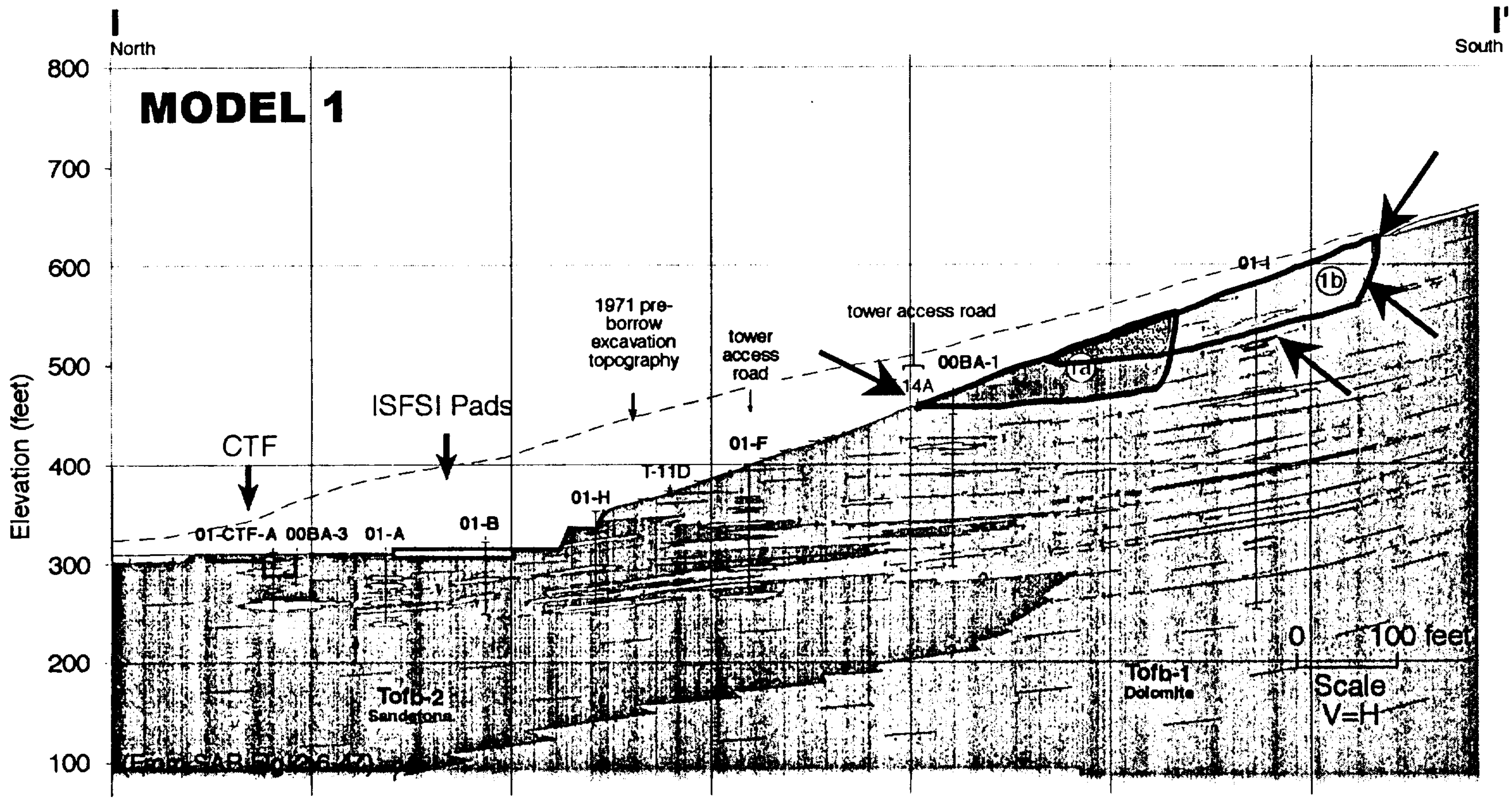
From SAR Figure 2.6-8

Analysis Cross Section I-I′



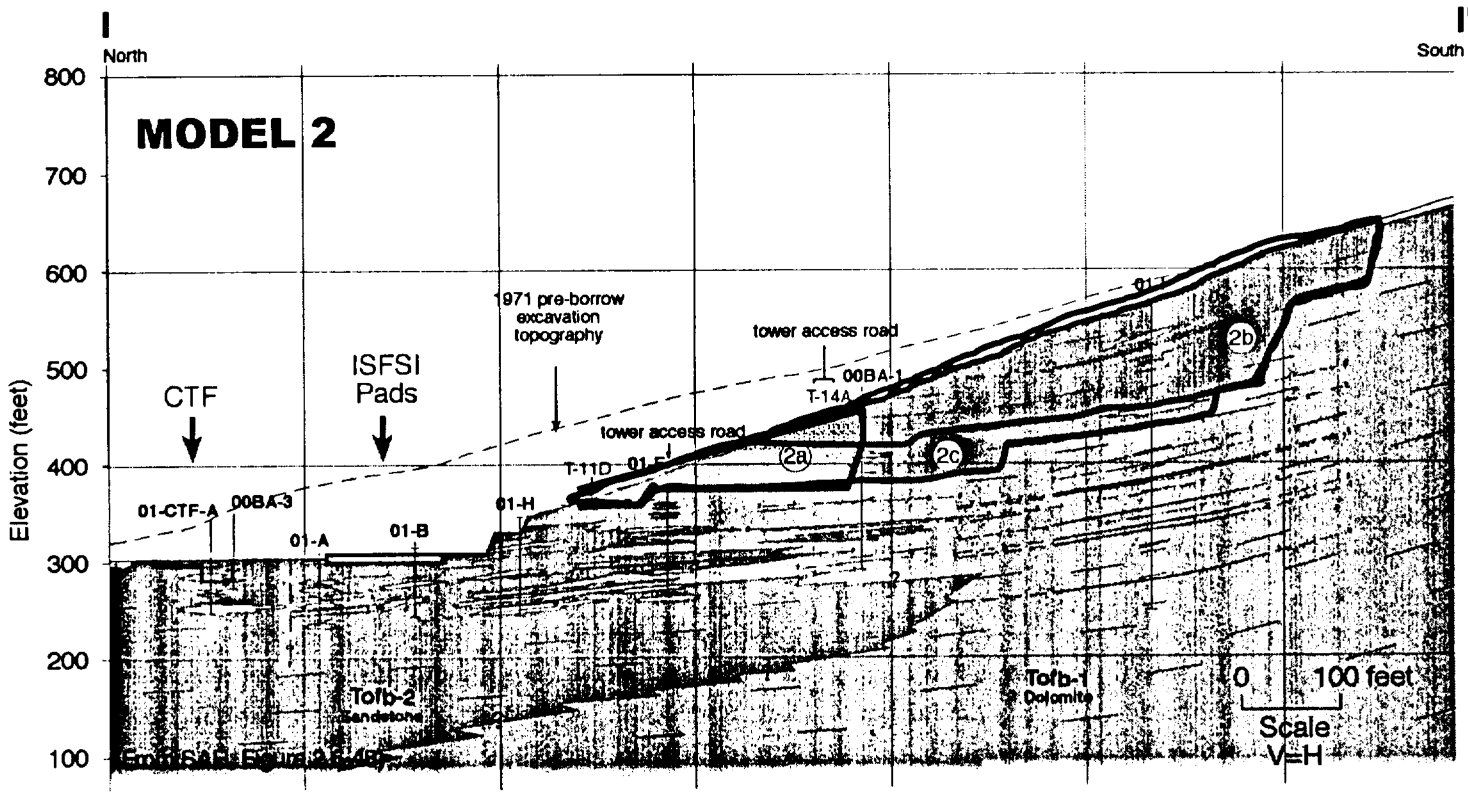
From SAR Figure 2.6-18

Potential Large-scale Rock Mass Model –Upper Slope



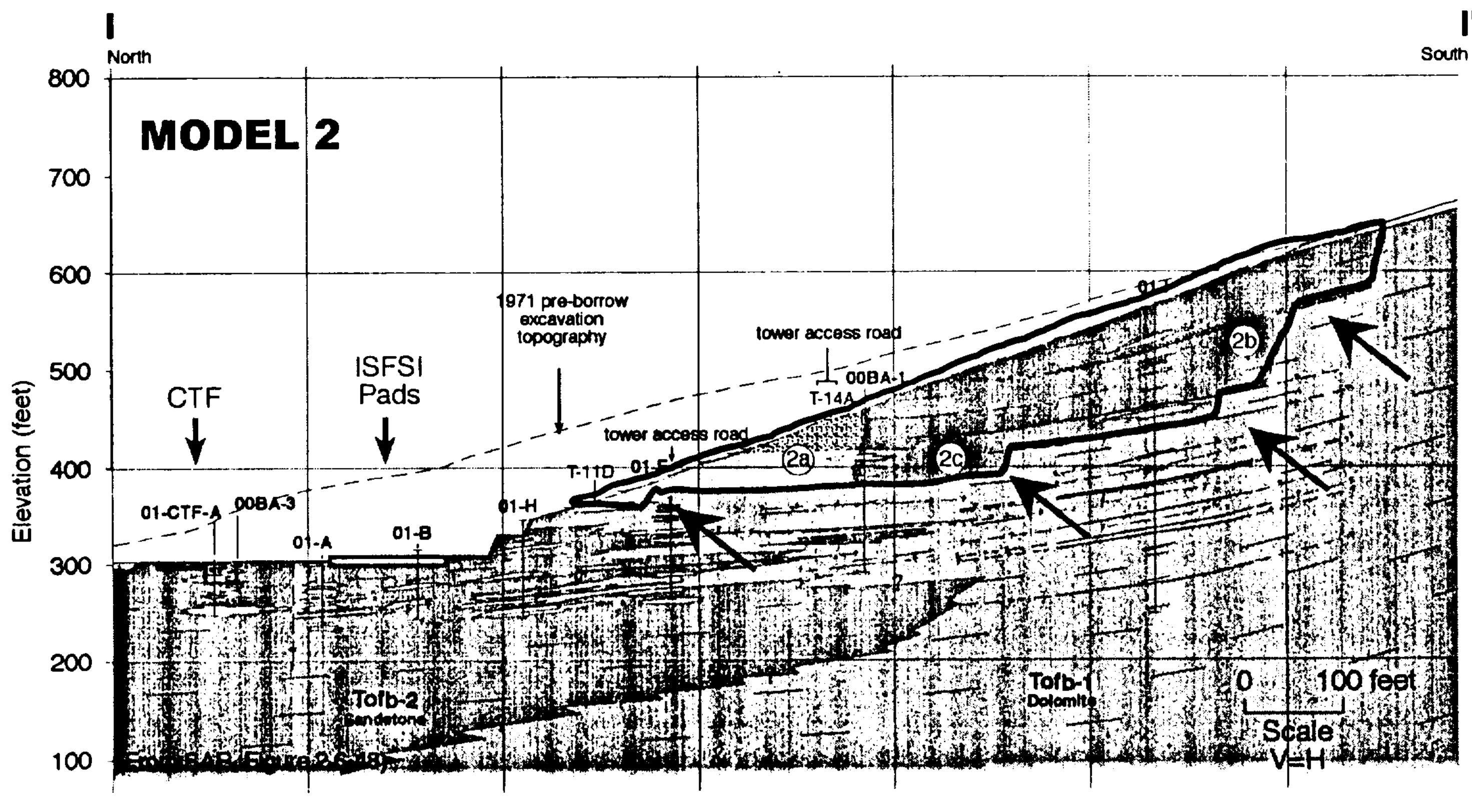
From SAR Figure 2.6-47

Potential Large-scale Rock Mass Model –Intermediate Slope



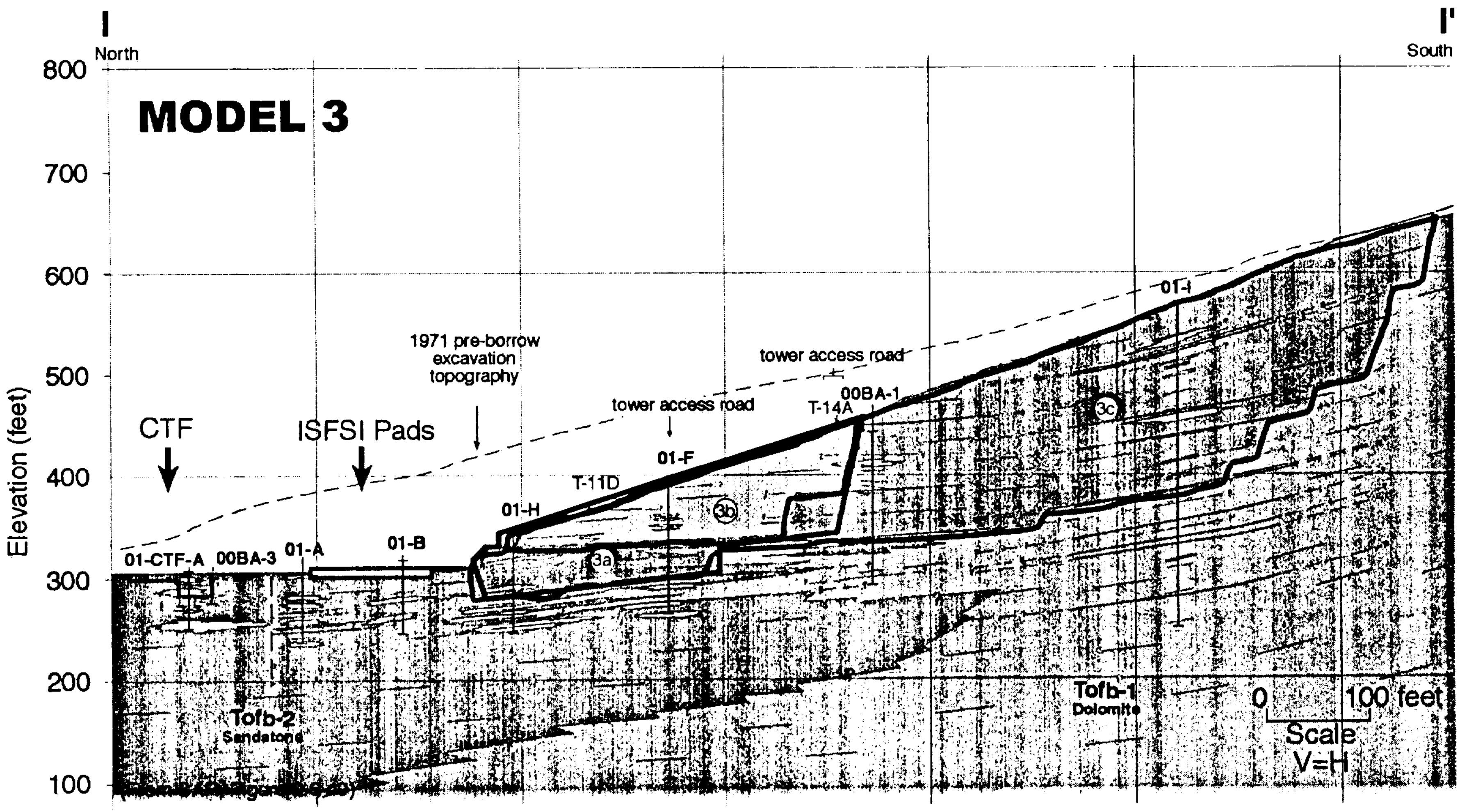
From SAR Figure 2.6-48

Potential Large-scale Rock Mass Model –Intermediate Slope



From SAR Figure 2.6-48

Potential Large-scale Rock Mass Model –Lower Slope



From SAR Figure 2.6-49

Static Slope Stability

- 2-D analysis using Spencer's method of slices
- Input:
  - Geometry
  - Material properties: unit weights, strengths
- Output:
  - Static factors of safety (F.S.)
  - Yield acceleration (k_y)

From SAR Section 2.5.1.2 and Calc Package GEO.DCPP.01.24

Slope Stability Theory

$$\textbf{F.S.} = \frac{\textbf{Resisting moment}}{\textbf{Driving moment}} = 1$$

Pseudo-static Slope Stability

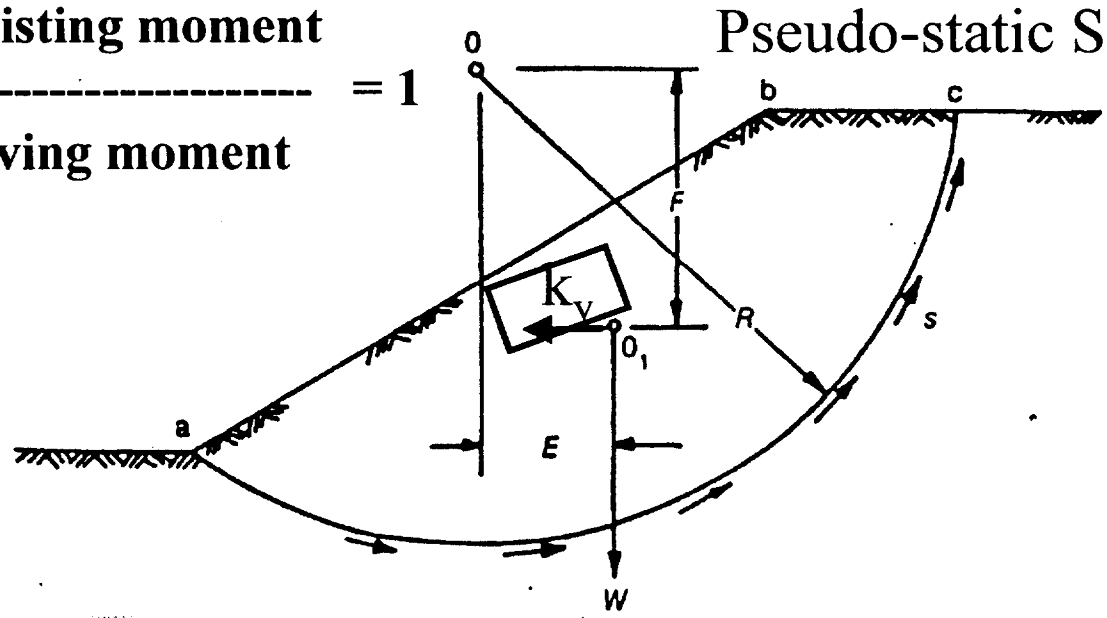


Fig. 2. Conventional method for computing effect of earthquake on stability of a slope (after Terzaghi, 1950)

Assumptions

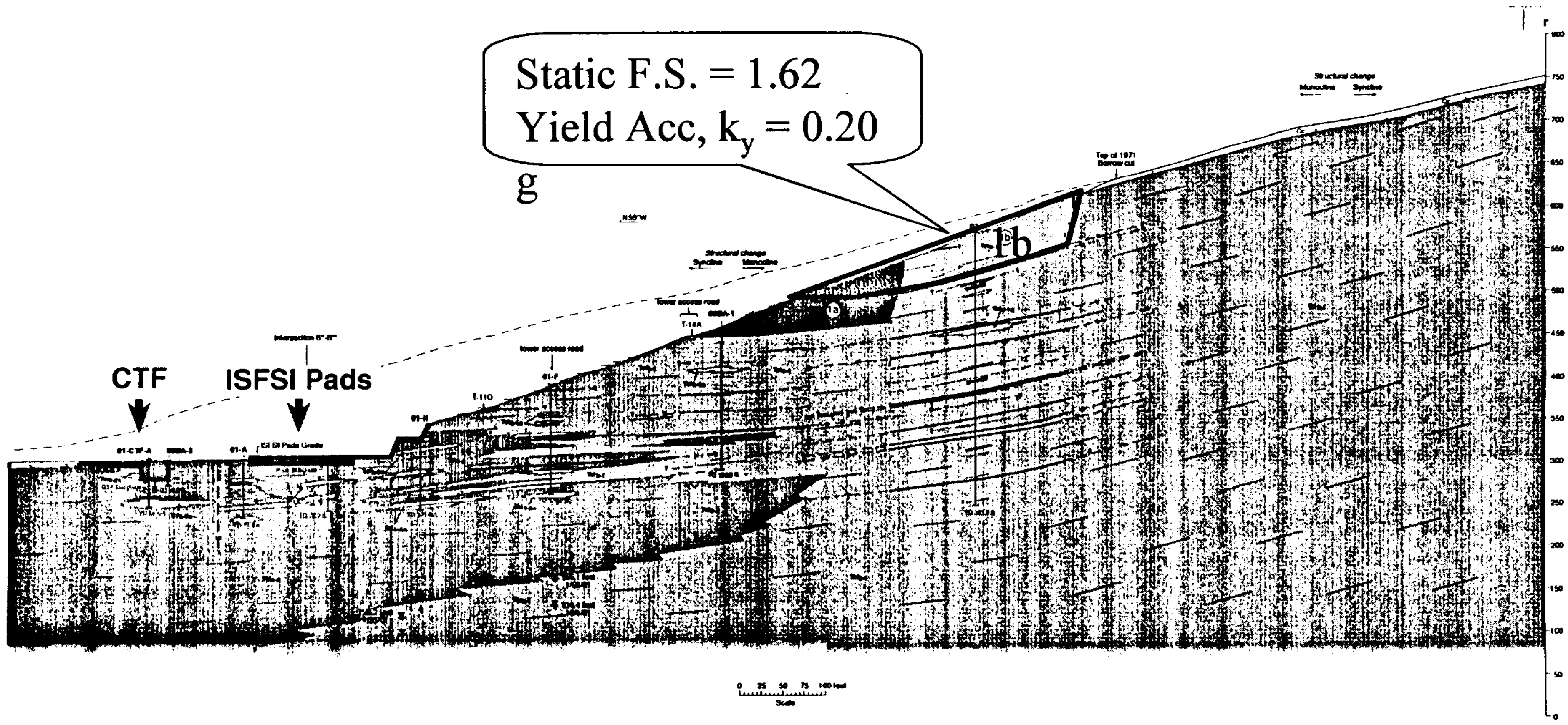
- Claybeds are saturated
- Tension cracks exist in the upper 20 ft
- Rock to rock contacts along the thin clay beds are neglected
- Lateral margins of potential slide masses are assumed to have no strength

From SAR Section 2.6.5.1.2.2

UTEXAS3 – Slope Stability

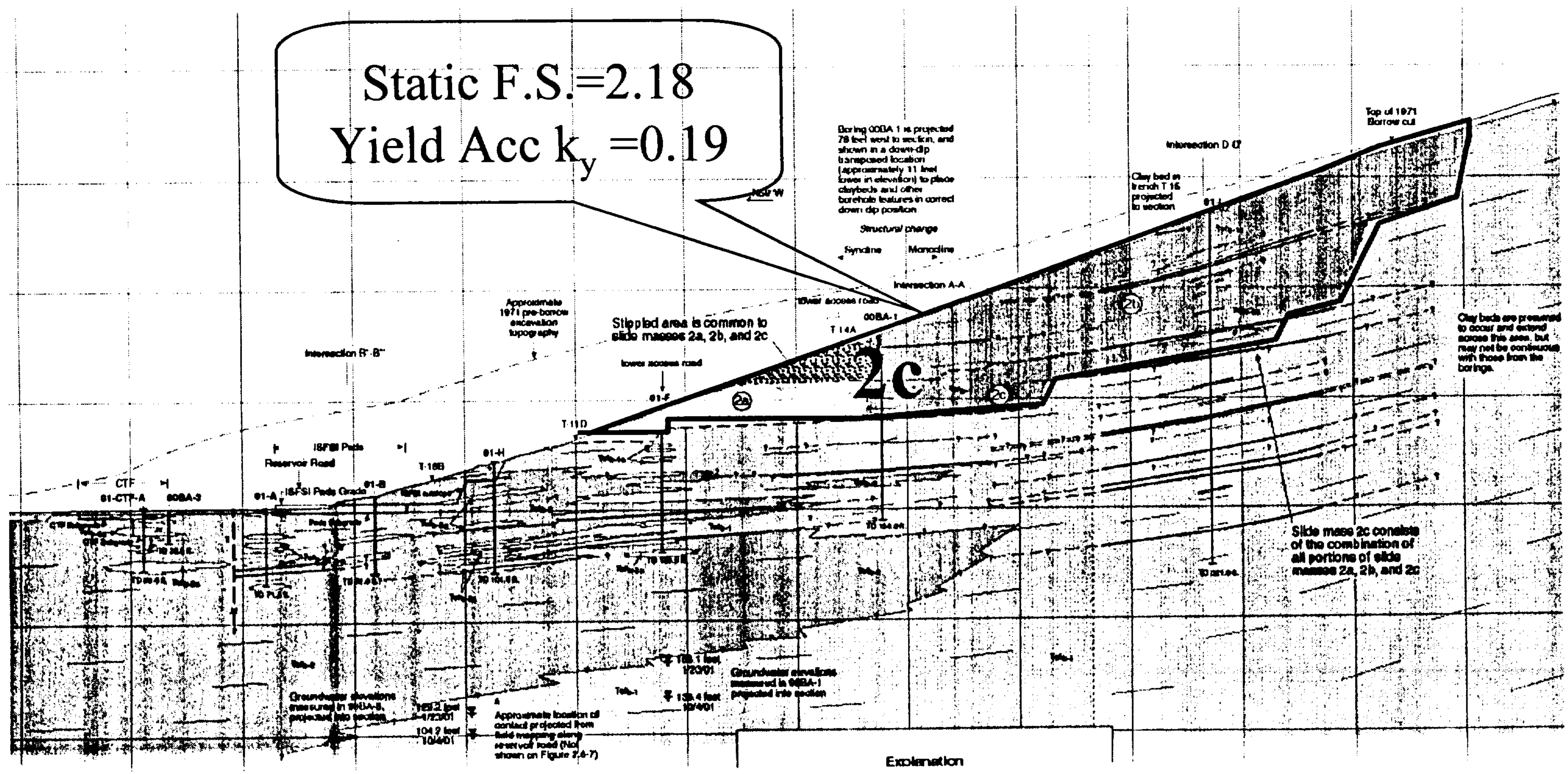
- Uses a 2-stage stability computation to evaluate the stability of slopes under seismic loading conditions (Duncan, Wright, and Wong, 1990)
  - 1st stage: computes the state of stress along the shear surface under long term loading conditions
  - 2nd stage: calculates undrained shear strength based on long term state of stress and performs slope stability analysis under seismic loading conditions

Cross Section I-I′ (shallow model)



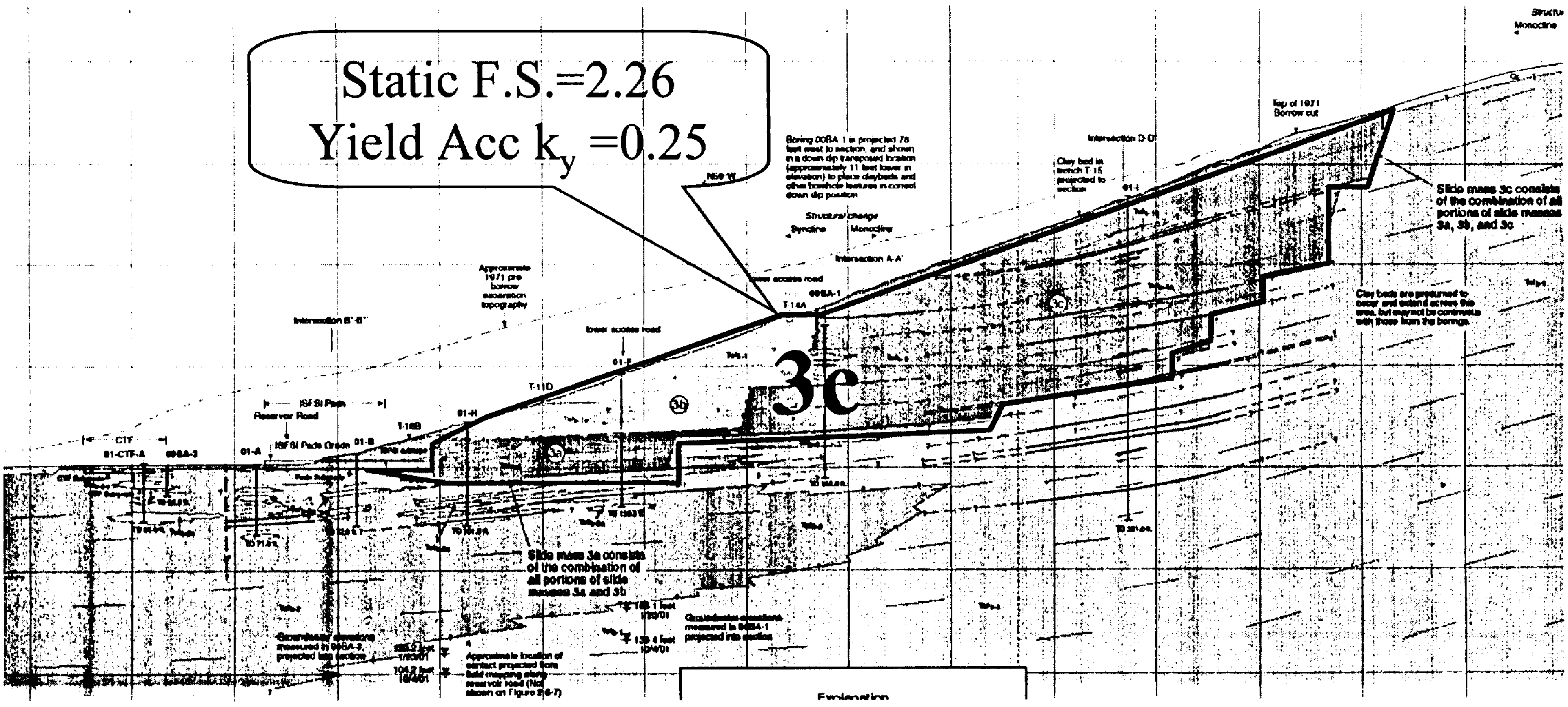
From SAR Figure 2.6-47

Cross Section I-I′ (medium depth model)



From SAR Figure 2.6-48

Cross Section I-I′ (deep model)



From SAR Figure 2.6-49

Results of Static Slope Stability

Slide Mass	Static F.S.	Yield Acc. (k_y)
1a	2.55	0.28
1b	1.62	0.20 ←
2a	2.55	0.31
2b	2.16	0.24
2c	2.18	0.19 ←
3a	2.86	0.44
3b	2.70	0.39
3c	2.26	0.25 ←
3c-1	2.38	0.28
3c-2	2.28	0.23

From SAR Table 2.6-3

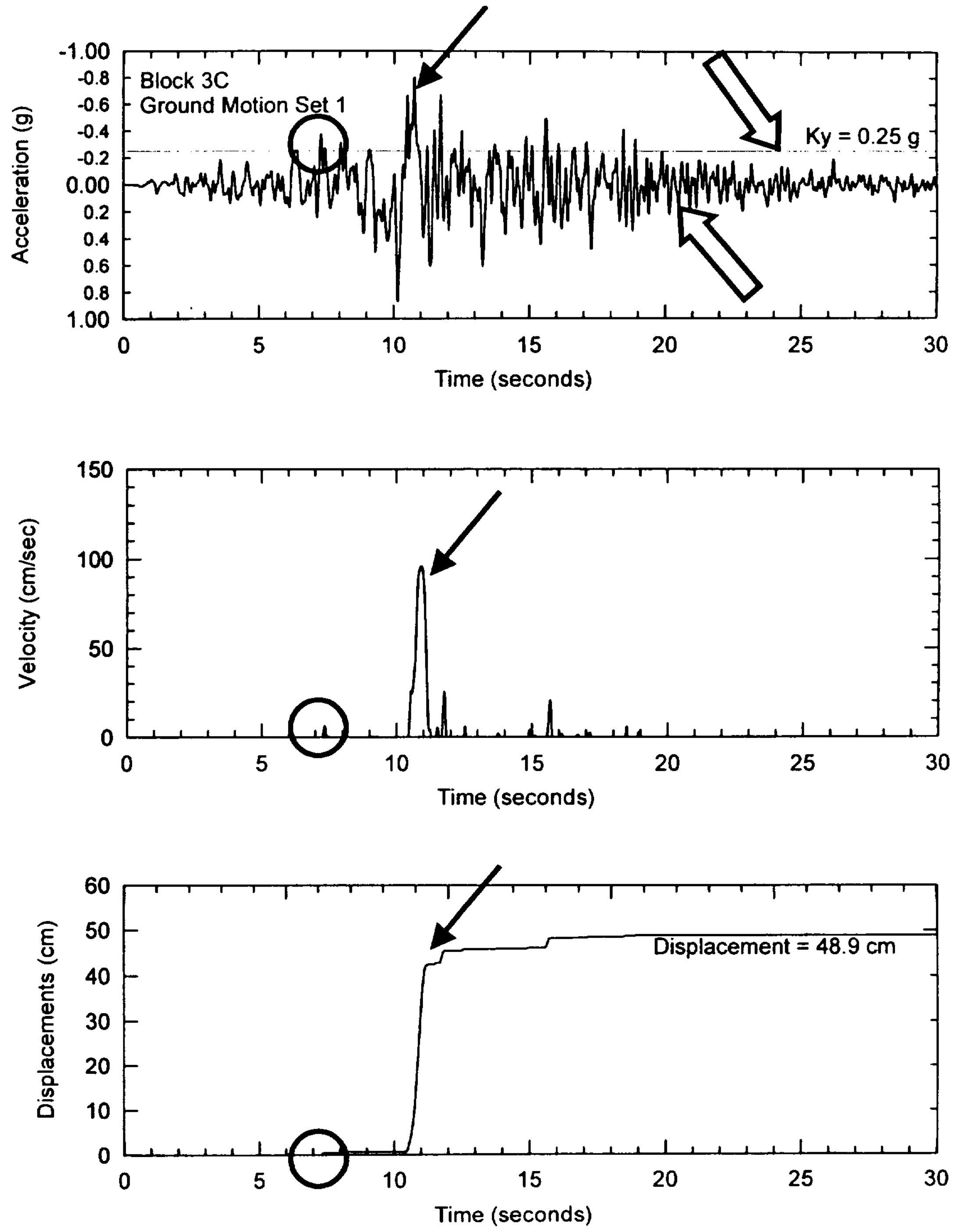
Methodology of Seismic Analysis

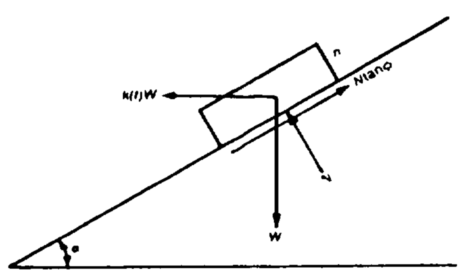
- Yield accelerations (k_y) determined from slope stability analysis
- Response of slide masses under seismic loading evaluated using 2-D finite element method
- Displacements of slide masses calculated using Newmark sliding block approach

QUAD4M – 2D Dynamic Response Analysis

- Input:
  - Unit weights
  - Shear wave velocities and damping values
  - Non-linear material properties
- Output:
  - Acceleration time histories of slide masses

Illustration of Newmark Sliding Block Displacement Calculation





NRC Request for Clarification

" ... the slope safety evaluation presented in the SAR, which was developed using the horizontal ground motion components without the vertical component, should be clarified ... "

Effects of Vertical Ground Motions on Sliding Analysis

- Inclination of slide planes (bedding planes)
  - Vertical motions more important for steeply dipping slide planes (greater than 30-40°)
  - ISFSI clay beds typically dip less than 15°
- Steepness of slope
  - Vertical motions more important for steeper slopes
  - ISFSI hillslope is about 3:1 (H:V) and the effect of steepness of slope is incorporated in the 2-D slope stability and dynamic response analyses

Effects of Vertical Ground Motions on Sliding Analysis

- Material strength properties
  - More important for sandy material
  - Less important for clayey material
  - ISFSI slide plane material is clay beds

Ground Motion Considerations for Sliding Block Analysis

- **Direction of slide mass movements:**
  - Occurs along claybeds
  - Claybeds are horizontal to sub-horizontal
  - Postulated slide mass movements are influenced by horizontal motions
- **Slide plane inclination:**
  - Limited effect on computed displacement (less than 10%) if the inclination is less than 20° based on Makdisi (1976) for sandy materials
  - For material similar to the ISFSI claybeds, the influence would be less
  - ISFSI claybeds typically dip less than 15°

Ground Motion Considerations for Sliding Block Analysis (cont'd)

- **Undrained shear strength of claybeds:**
  - Controlled by long term overburden pressure
  - Relatively insensitive to seismic loadings
- **Peak arrival time:**
  - Arrival time of horizontal peak is typically 1 to 3 seconds behind arrival of vertical peak based on near field recordings
  - During strong horizontal shaking, the energy (as measured by Aries Intensity) on the vertical component is typically 10% to 30% of the energy on the horizontal component

Ground Motion Considerations for Sliding Block Analysis (cont'd)

- **Standard of practice for seismic design of dams:**
  - Evaluation of permanent displacement of embankment dams under seismic loading is based on horizontal component of the design motion (USBR, 1989)
- **Recent studies**:
  - Study at Cal Tech indicated that vertical ground motions have limited impact on block movements based on numerical analysis and physical modeling Yan, et al. (1996)

Preliminary Site-Specific Study

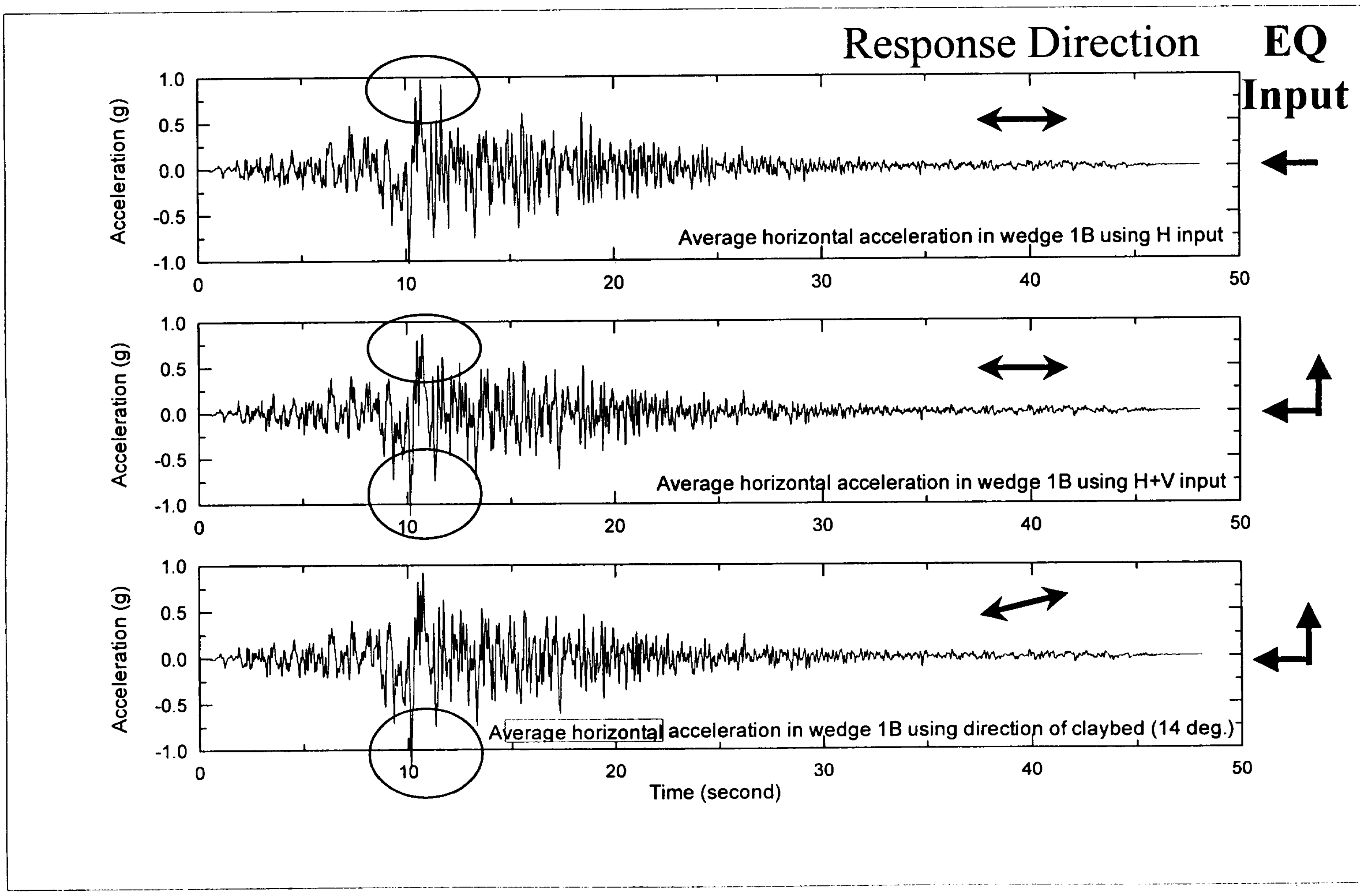
- Evaluate effect of vertical motions on computed yield acceleration
- Evaluate effect of vertical motions on slide mass responses
- Incorporating vertical ground motions resulted in displacements varying less than 10% from calculations based on horizontal component alone

Effects of Vertical Seismic Coefficient (k_v) on Yield Acc (k_y)

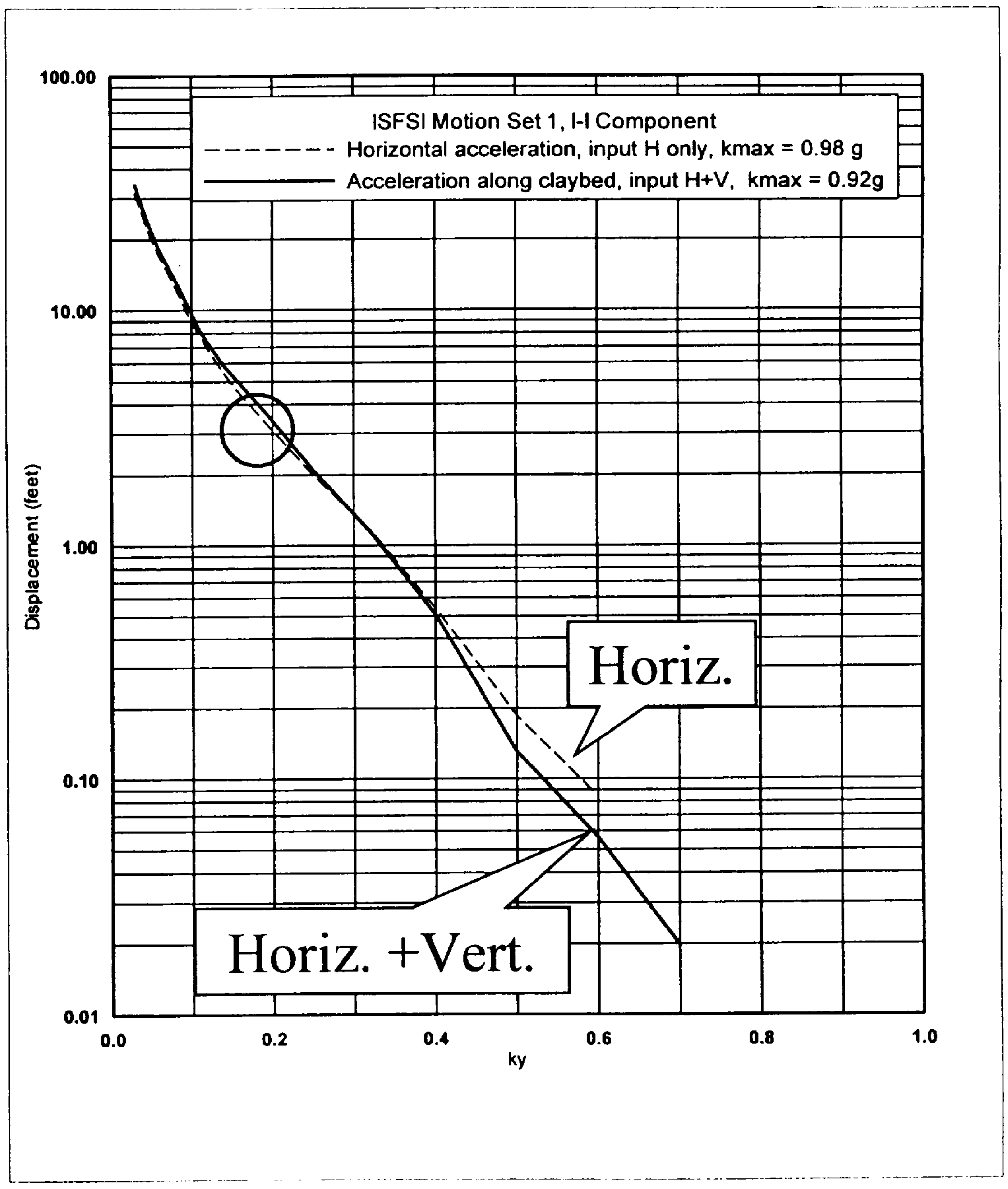
	k_v / k_h	k_y	
$-k_v$ up			
	-0.8	0.23	
	-0.6	0.22	
	-0.4	**0.20**	Zone of applicability
	-0.2	**0.19**	
	+0.0	**0.19**	
	+0.2	**0.18**	
	+0.4	**0.17**	
	+0.6	0.17	
$+k_v$ down	+0.8	0.16	

Based on hand calculation of slide mass 1B

Response of Block 1B



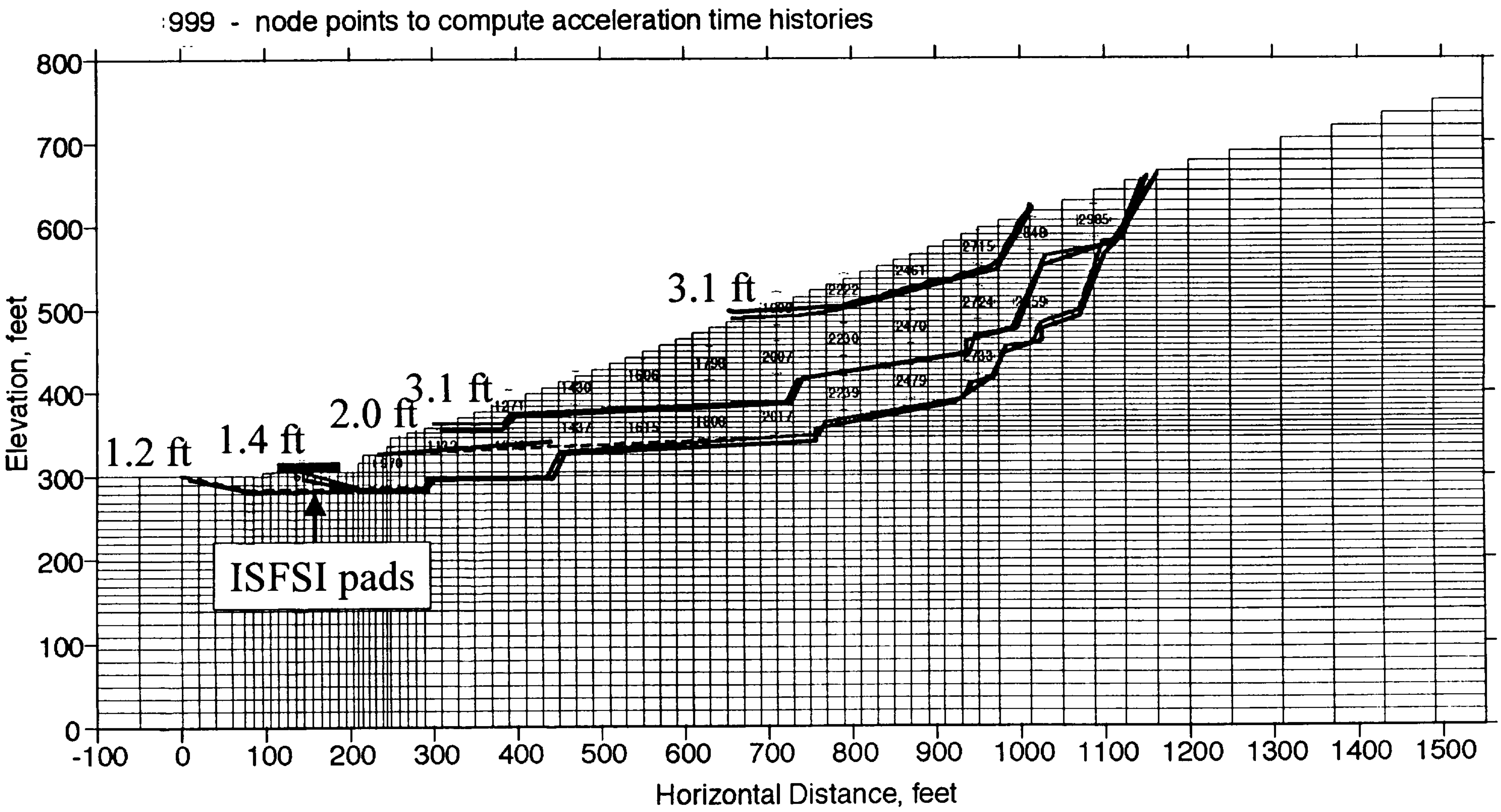
Vertical Ground Motion Effects on Computed Displacements



Conclusions on the Effects of Vertical Ground Motions

- Effect on clay bed shear strength
  - Minimal to none
- Effect of inclination of potential slide plane
  - Minimal
- Effect on computed horizontal response of slide masses
  - Minimal
- Overall effect on computed displacements
  - $\pm 10\%$

Potential Seismic Induced Displacements on Clay Beds



Mitigation Measures for Slide Masses

- Set back
  - 25 ft wide bench between cutslopes
  - 40 ft clearance between edge of ISFSI pads and toe of cutslope

- Debris fences

Conclusions

- The stability of the hillslope above the ISFSI pads was analyzed and the slopes have ample factors of safety under static conditions.
- The hillslope above the ISFSI site may experience small displacements when exposed to the design-basis earthquakes.

Conclusions (cont'd)

- The maximum seismic induced displacements could potentially be about 3 feet on the upper slope to about 1 to 2 feet on the lower slope.
- Mitigation measures will be implemented to minimize effects of the small displacements and protect the ISFSI facilities to perform their intended design functions.

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Diablo Canyon Independent Spent Fuel Storage Installation

Transport Route Stability and Displacement Analyses

Robert White

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April 11, 2002

Transport Route Slope Stability and Displacement Analyses Steps

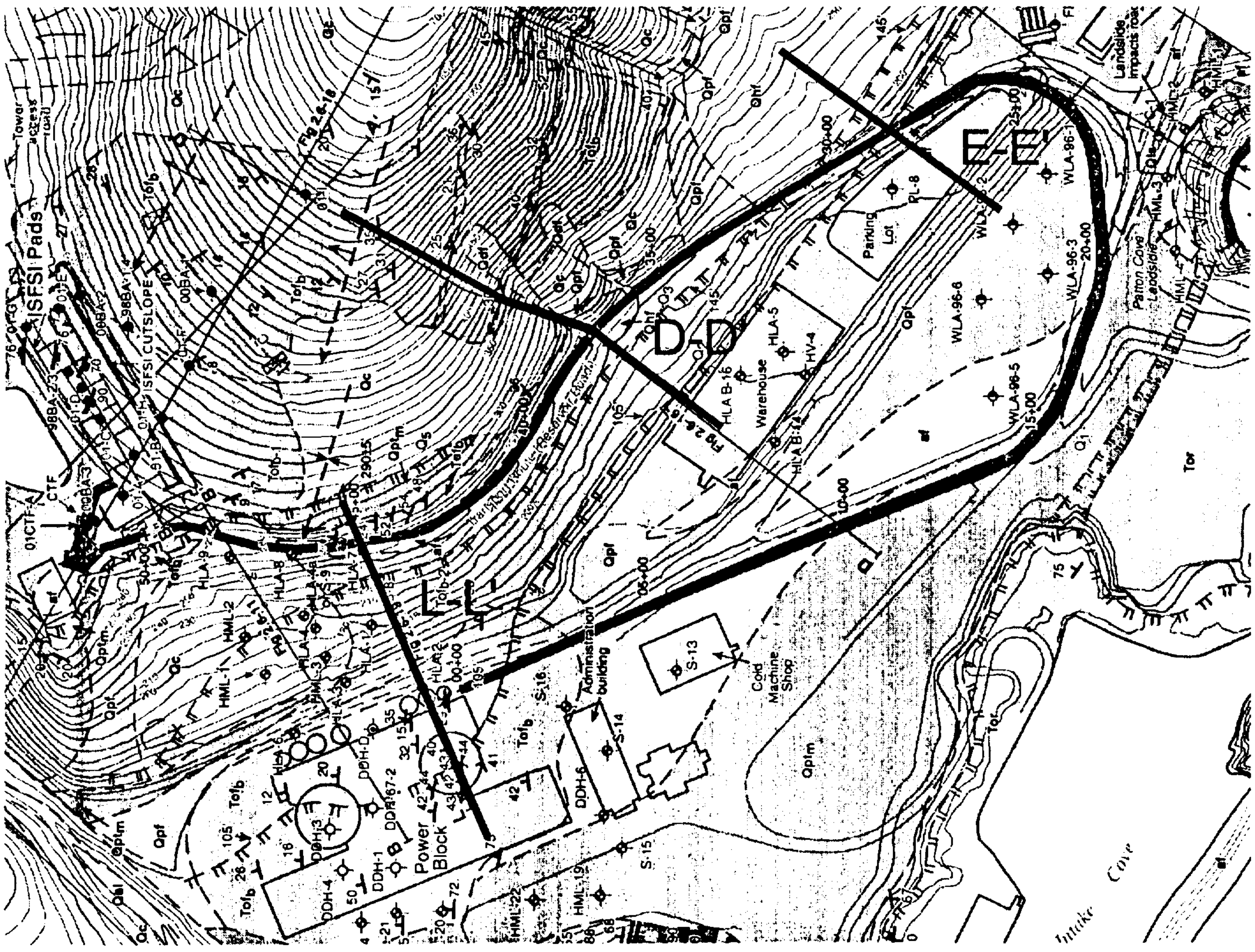
1. Locate critical slide mass with minimum factor of safety. Determine yield acceleration for critical slide mass.
2. Determine seismic coefficient time history for critical slide mass.
3. Determine potential earthquake-induced displacement of critical slide mass.

From SAR 2.6.5.4.2; GEO.DCPP.01.28, 29, AND 30

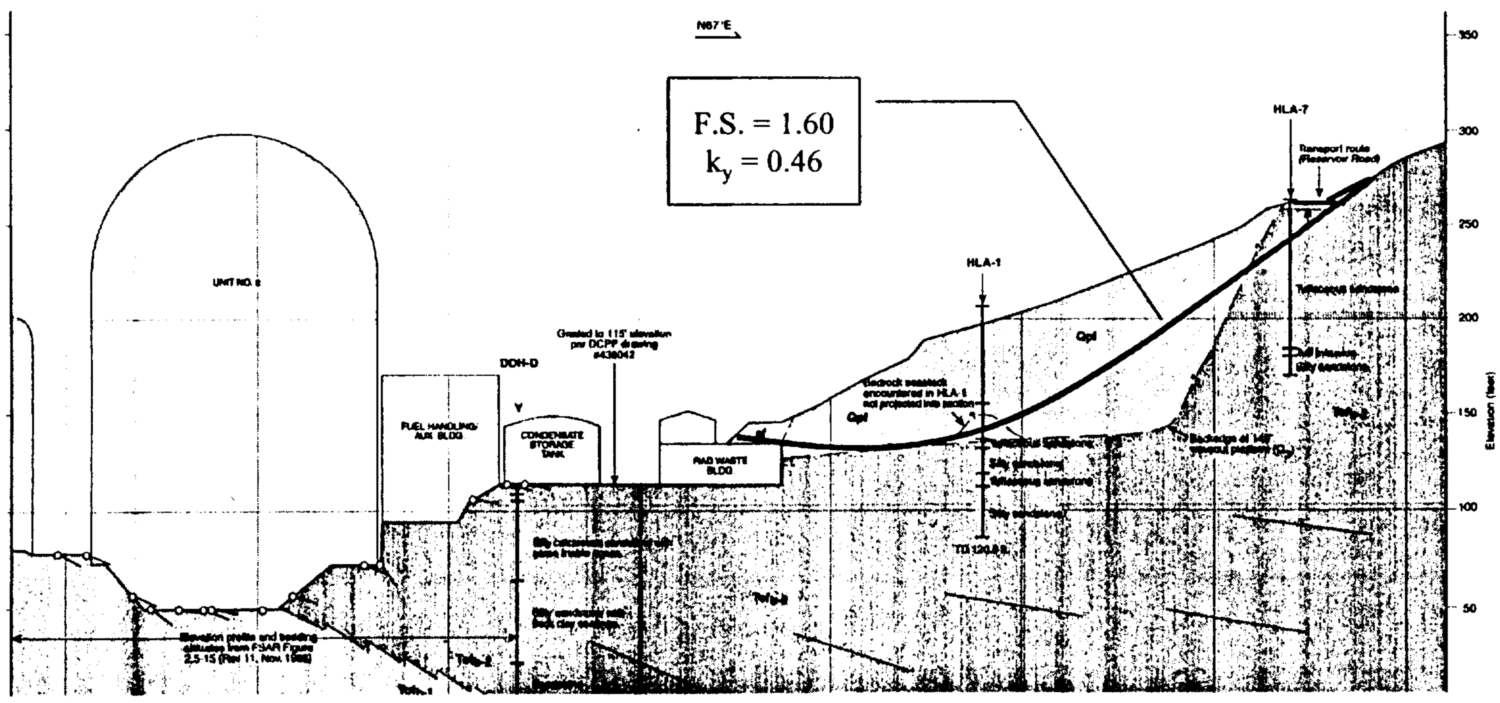
Stability and Yield Acceleration Analysis of Critical Slide Masses

- Three representative sections along the transport route were selected
- Affect of transporter load also evaluated

Analytical Sections



Slope Stability Analyses of Slide Masses



Finite Element Sections

- Finite element meshes for Sections L-L' and E-E' were prepared.

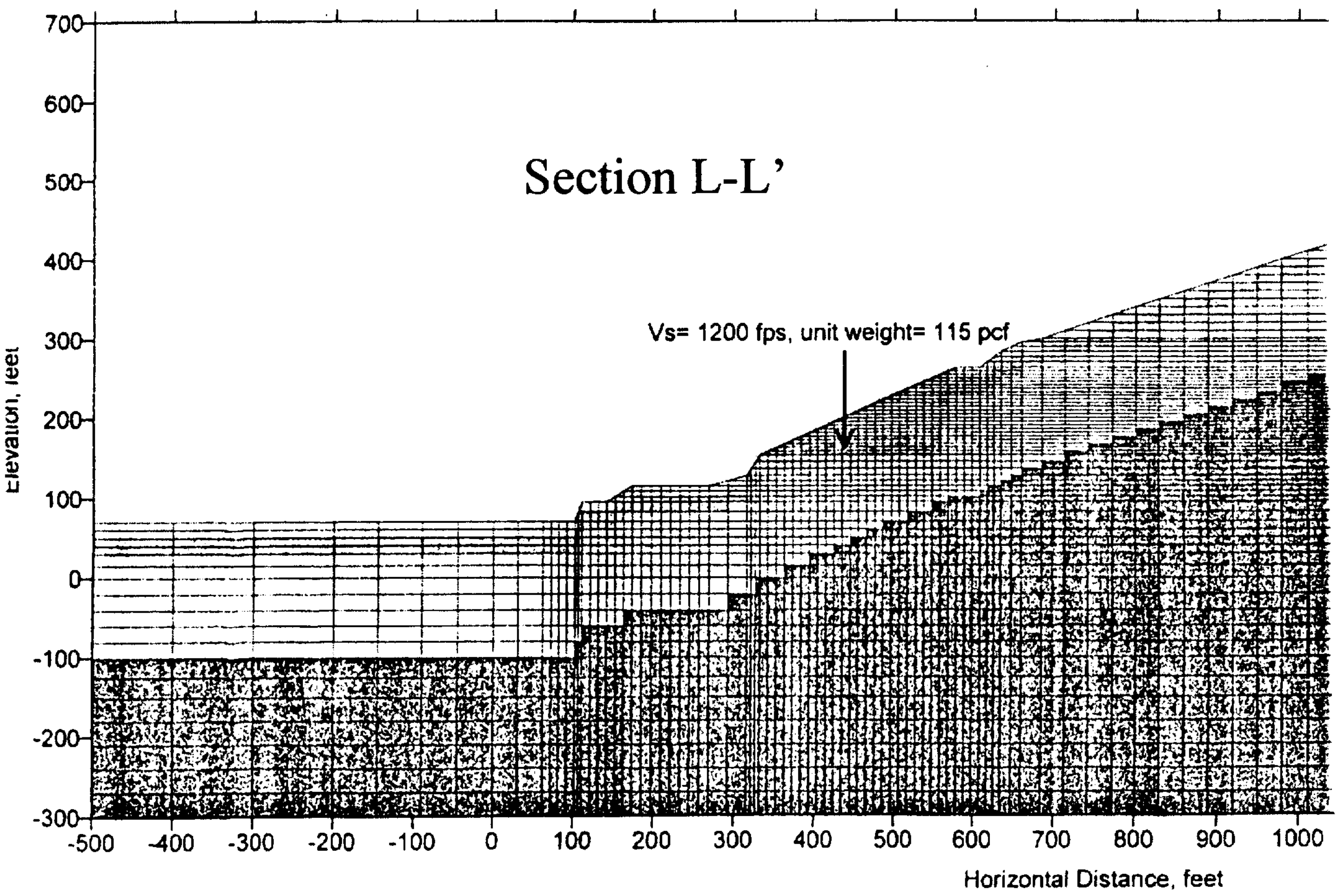
- A finite element mesh for Section D-D' was not prepared, as it is similar in configuration to Section E-E'.

Seismic Coefficient Time Histories of Slide Masses

- Finite element meshes for Sections L-L' and E-E' were prepared.

- A finite element mesh for Section D-D' was not prepared, as it is similar in configuration to Section E-E'.

Seismic Response Analyses Slide Masses



[From Figure 1, GEO.DCPP.01.29]

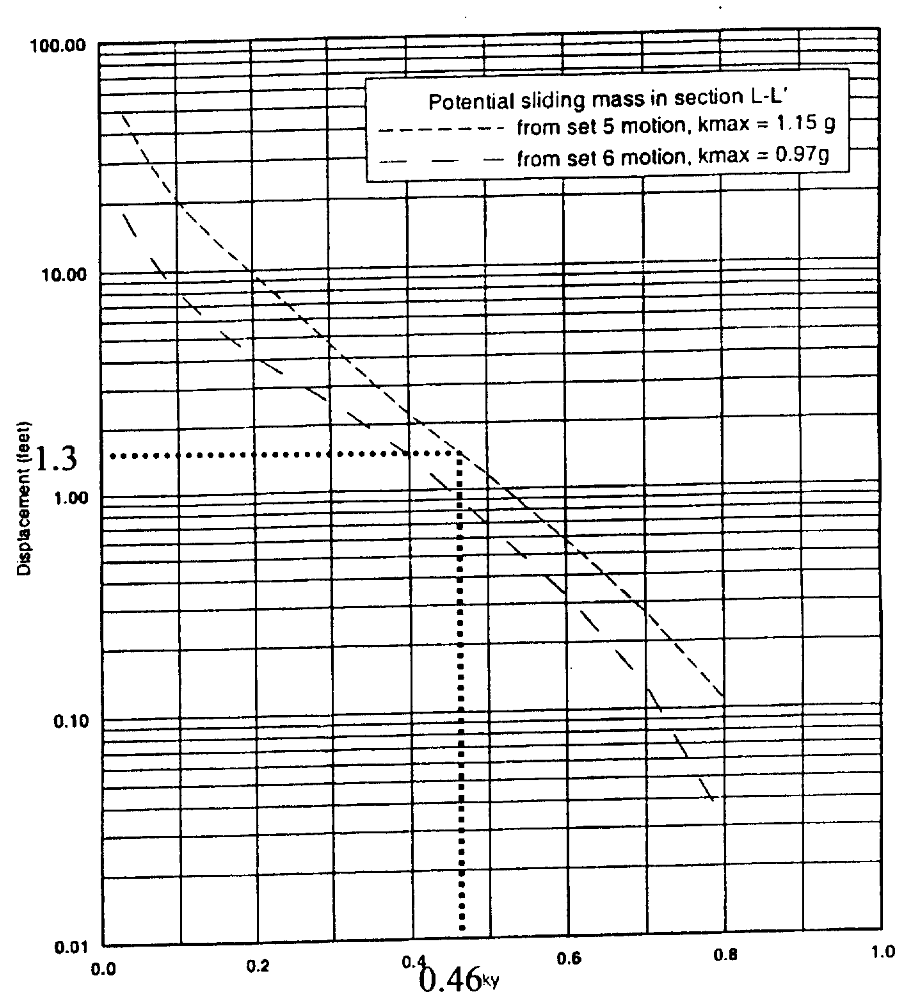
Seismic Coefficient Time Histories of Critical Slide Masses

- ILP ground motions were rotated to the direction of Sections L-L' and E-E' and input into the finite element program.
- The seismic coefficient time histories for the critical slide masses were obtained by averaging multiple nodal point time histories within the respective masses.

Earthquake-Induced Displacements of Critical Slide Masses

- The Newmark sliding block analysis procedure was used to estimate the potential displacements of the critical slide masses using the seismic coefficient time histories to estimate the potential slide mass movements.
- Potential displacements of the critical slide masses in the three sections analyzed range from 0.5 to 1.3 feet.

Transport Route Displacement Analyses



From Figure 6, GEO.DCPP.01.30

Conclusions

- Three representative sections along the transport route were evaluated for static and seismic loading conditions
- The sections are stable under transporter loads
- Displacements of 1.3 feet or less were calculated for slide masses subjected to the ILP ground motion

NRC/PG&E Open Meeting, San Francisco, CA
Diablo Canyon Independent Spent Fuel Storage Installation

Stability Analysis of Cutslopes

Jeff Bachhuber
Engineering Geologist
William Lettis & Associates

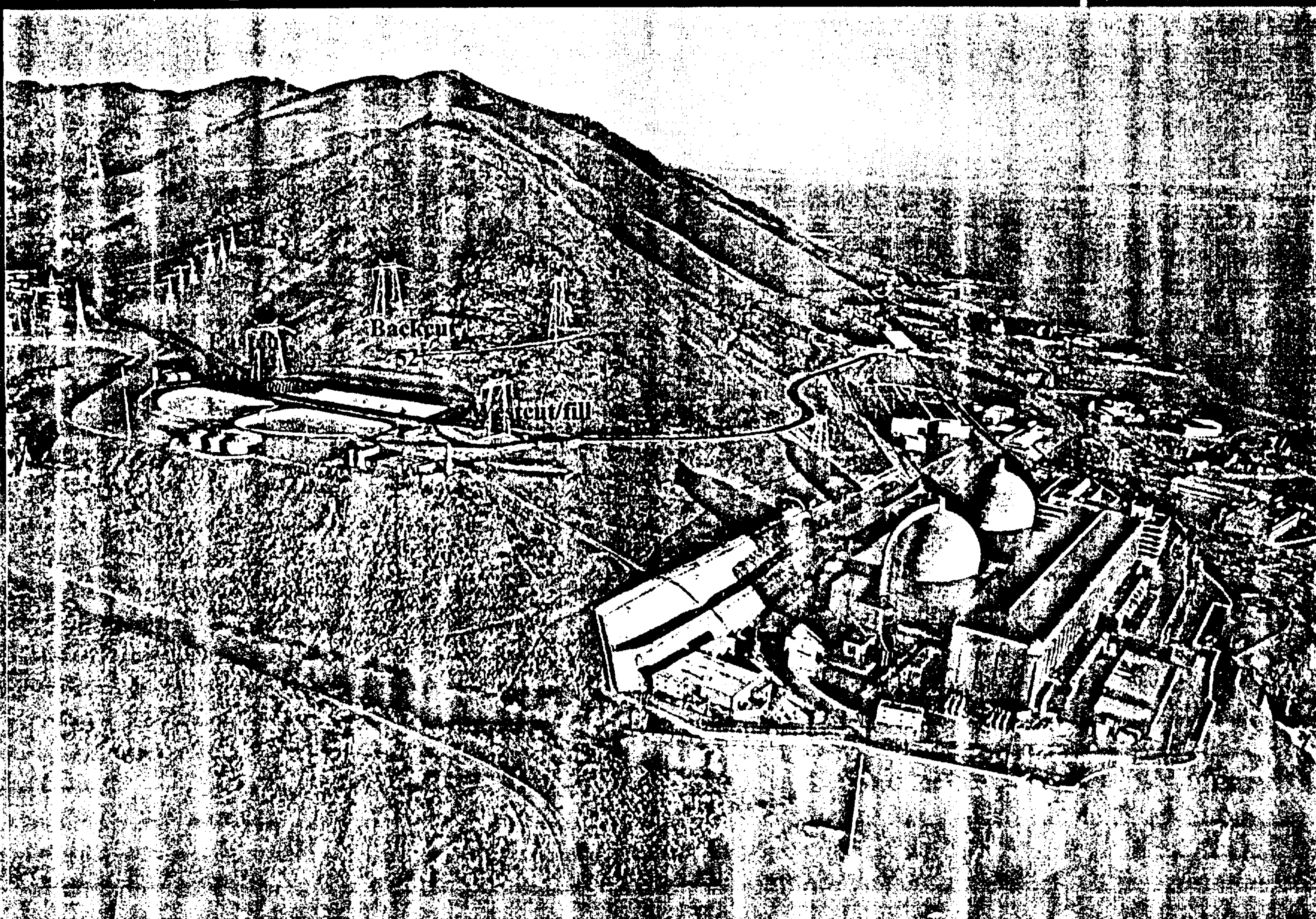
PG&E

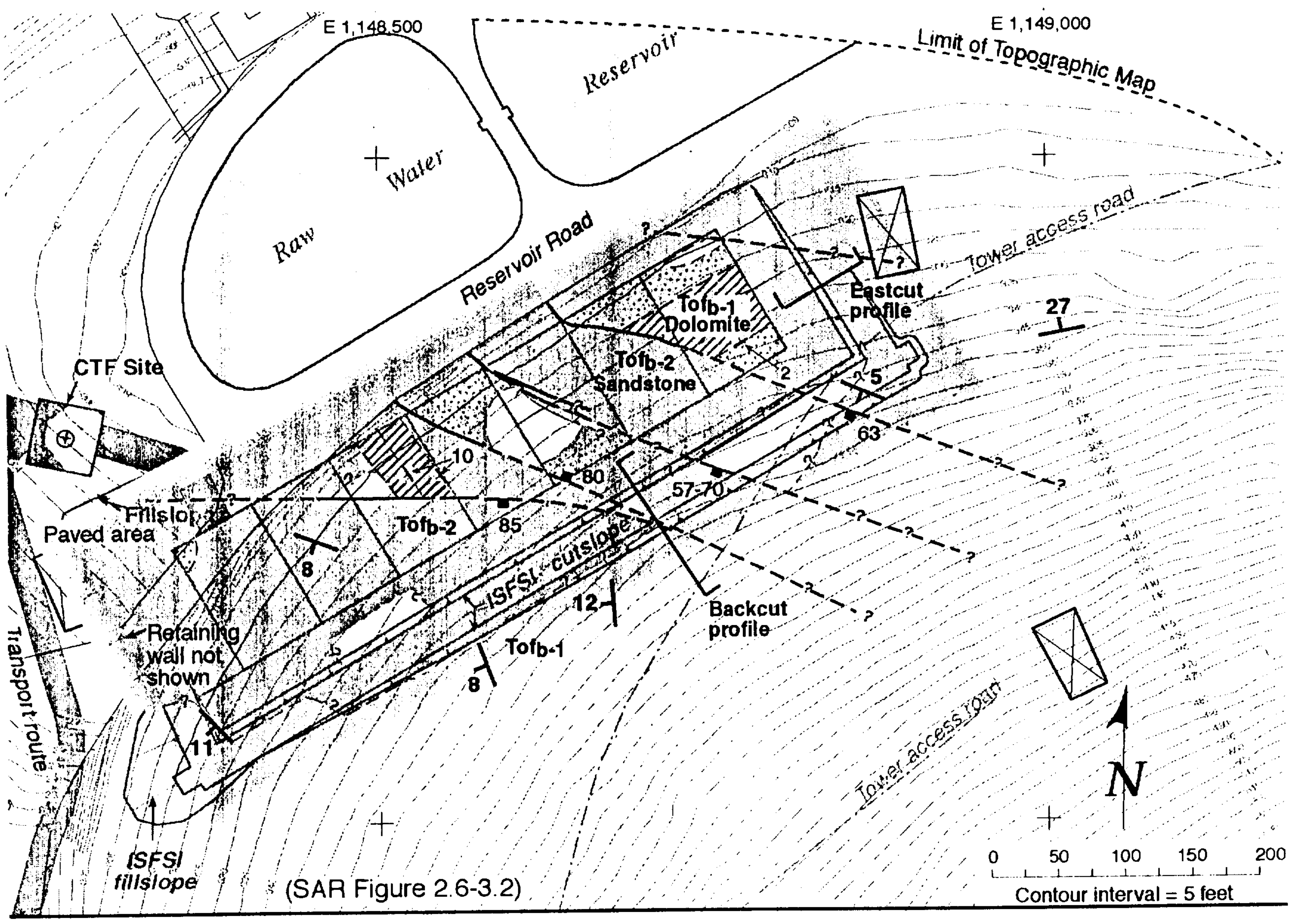
April 11, 2002

Purposes

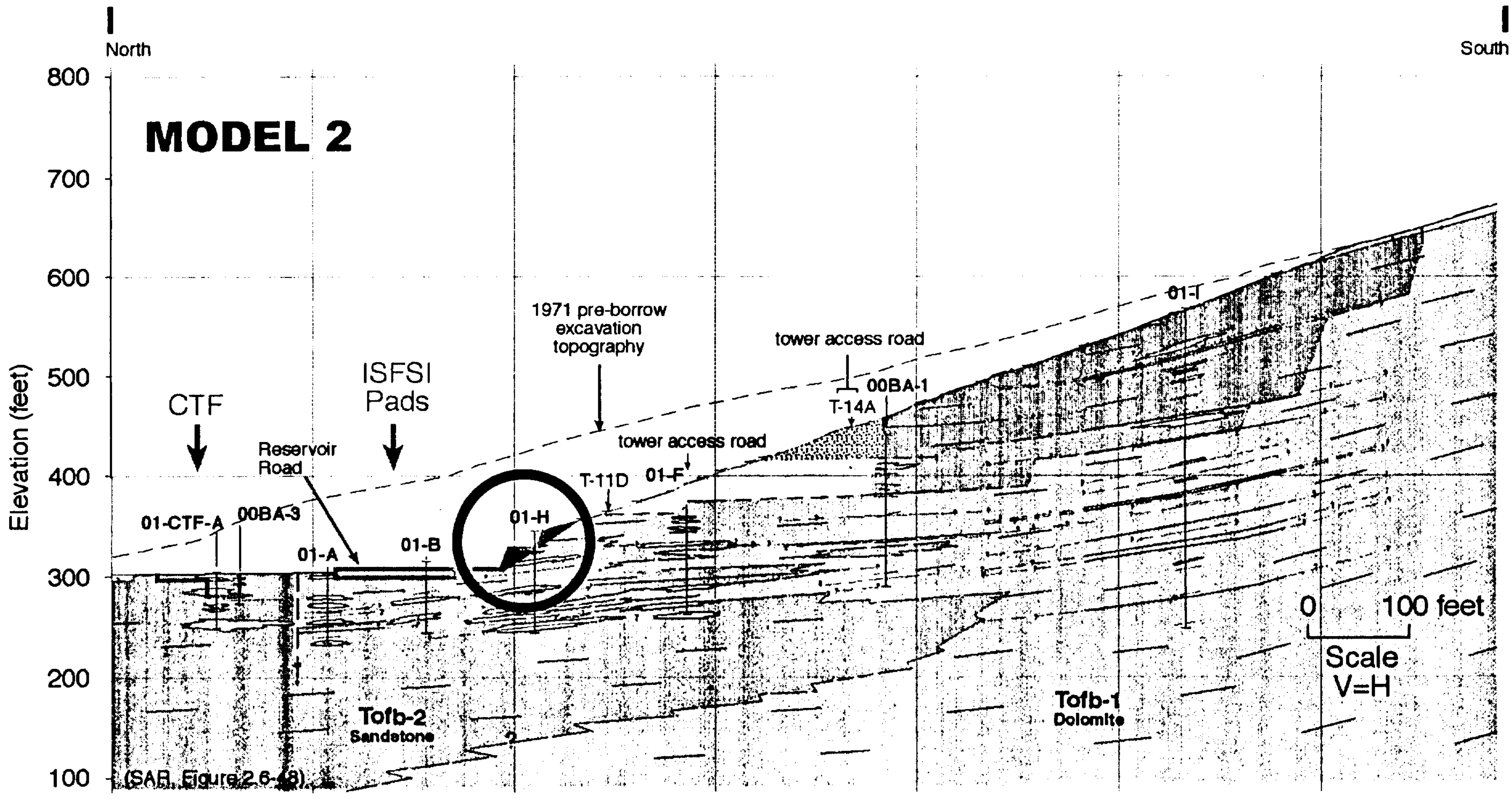
- Evaluate static and dynamic stability of proposed ISFSI pad cutslopes against possible smaller-scale rock block (wedge) failures
- Develop conceptual cutslope rock anchor support

Location of ISFSI Cutslopes

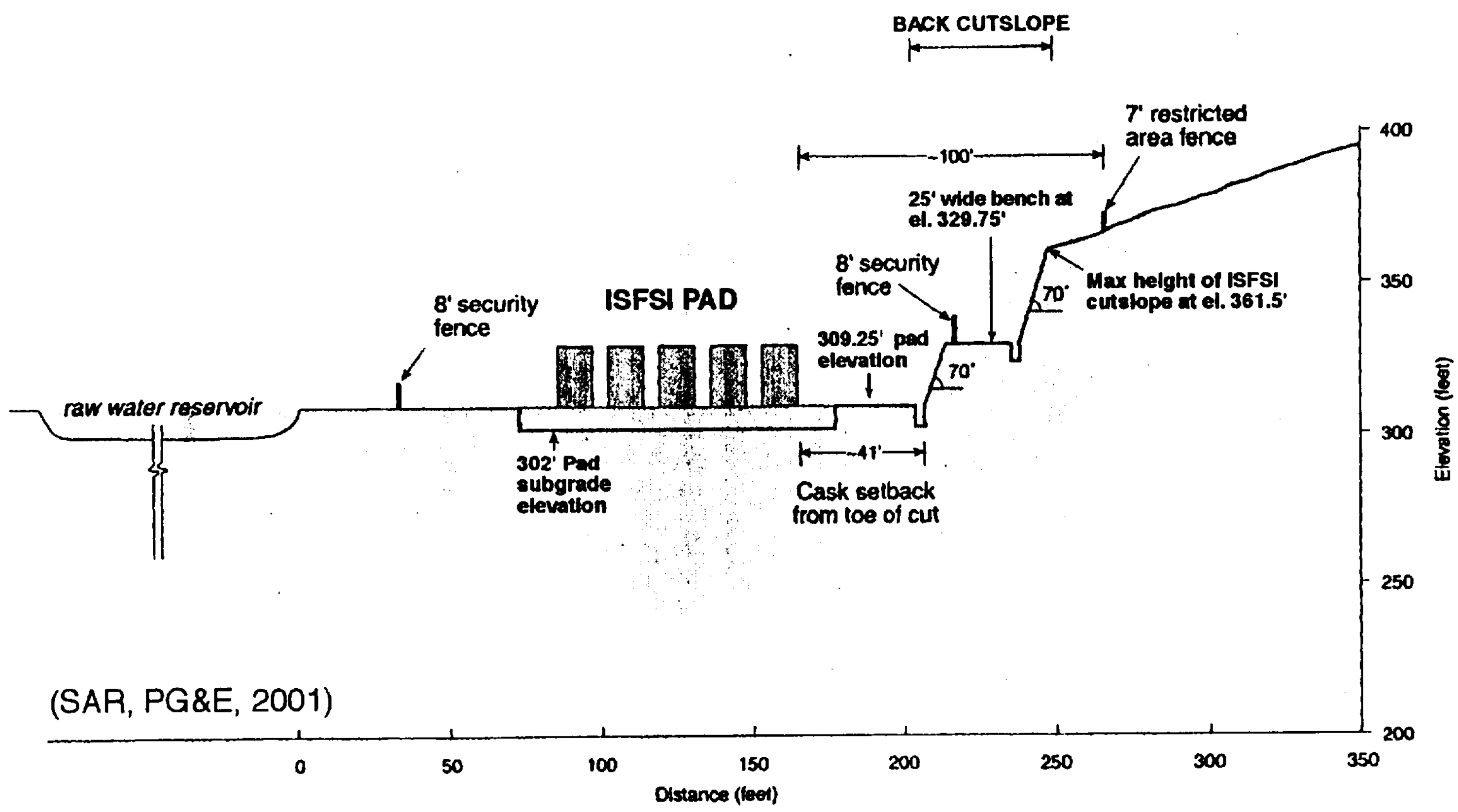




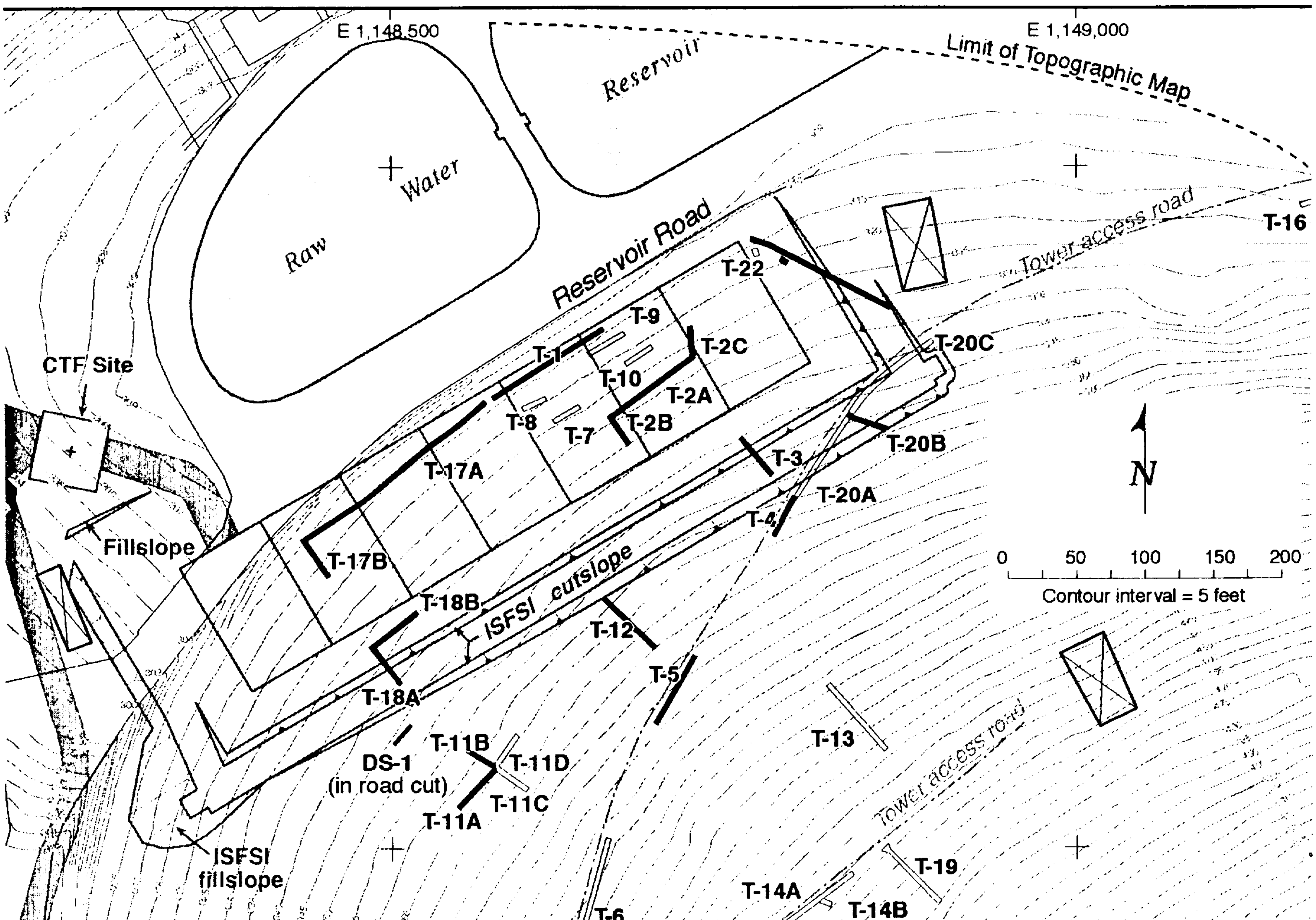
(SAR Figure 2.6-3.2)



I
North
I'
South
MODEL 2
Elevation (feet)
800
700
600
500
400
300
200
100
CTF
ISFSI
Pads
Reservoir
Road
1971 pre-borrow
excavation
topography
tower access road
tower access road
01-CTF-A
00BA-3
01-A
01-B
01-H
T-11D
01-F
T-14A
00BA-1
01-I
0
100 feet
Scale
V=H
Tofb-2
Sandstone
Tofb-1
Dolomite
(SAR, Figure 2.6-18)



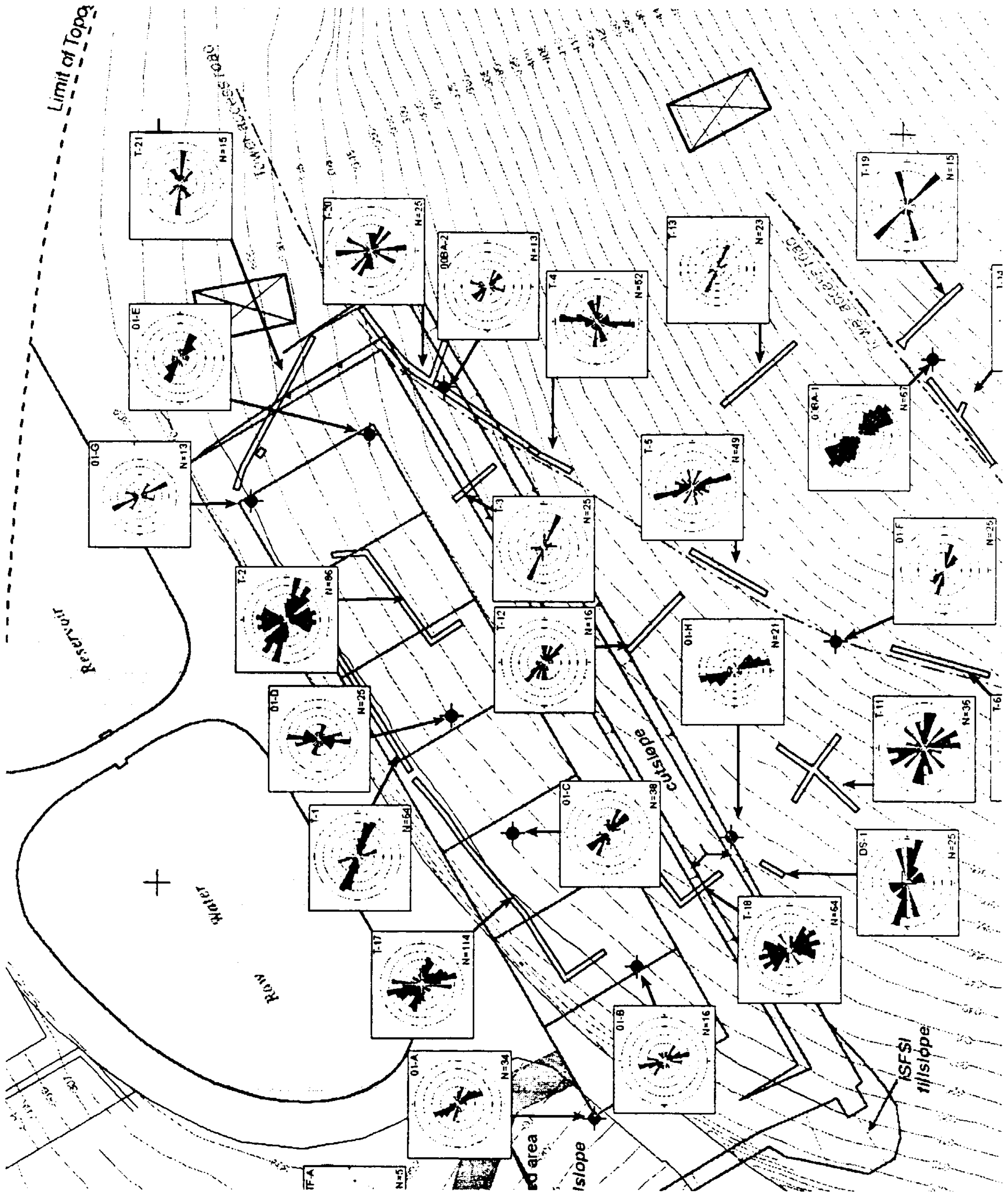
BACK CUTSLOPE
~100'
7' restricted
area fence
25' wide bench at
el. 329.75'
8' security
fence
Max height of ISFSI
cutslope at el. 361.5'
70°
70°
8' security
fence
ISFSI PAD
309.25' pad
elevation
raw water reservoir
302' Pad
subgrade
elevation
~41'
Cask setback
from toe of cut
(SAR, PG&E, 2001)
0
50
100
150
200
250
300
350
Distance (feet)
400
350
300
250
200
Elevation (feet)



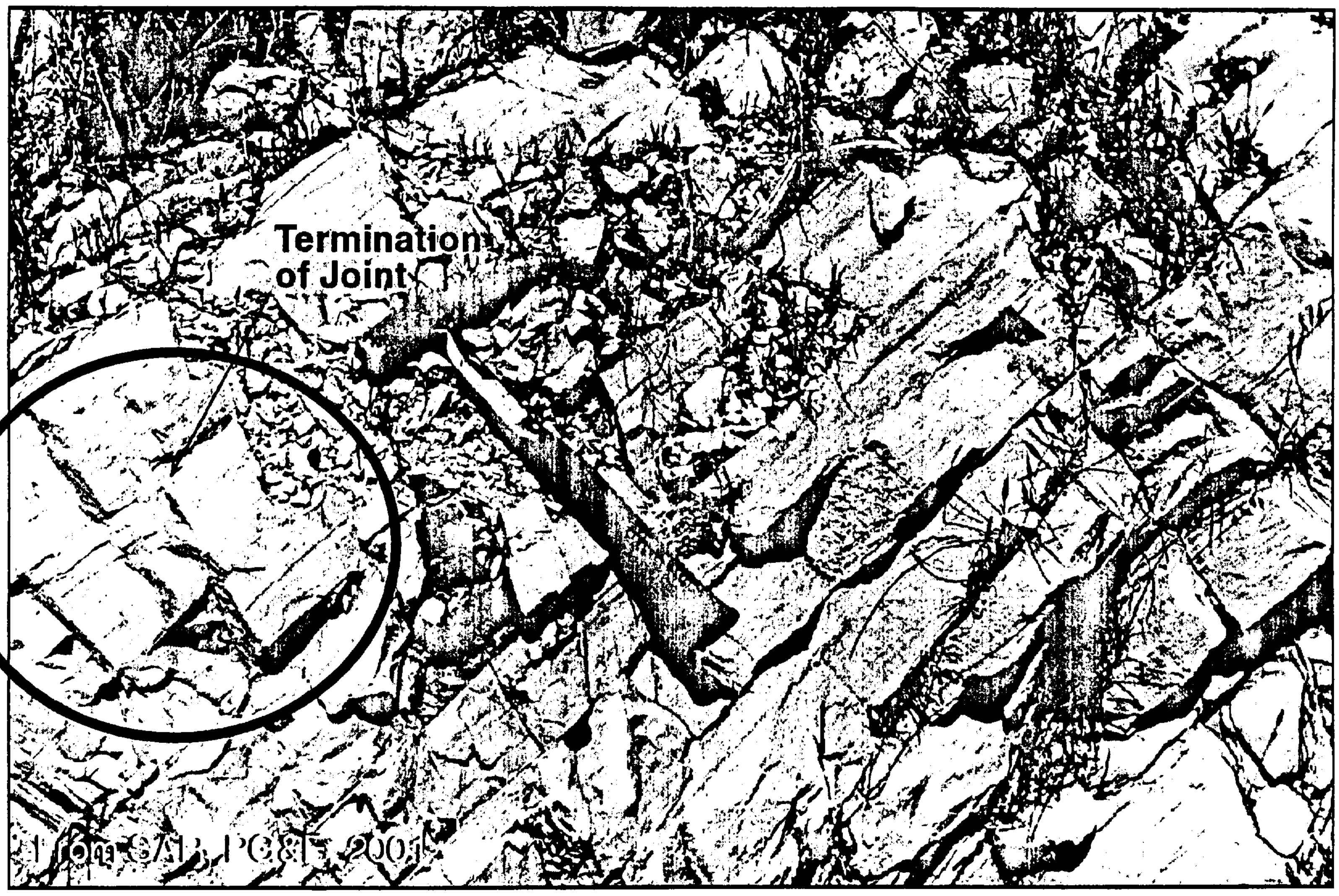
(Data Report F, Figure F-1)

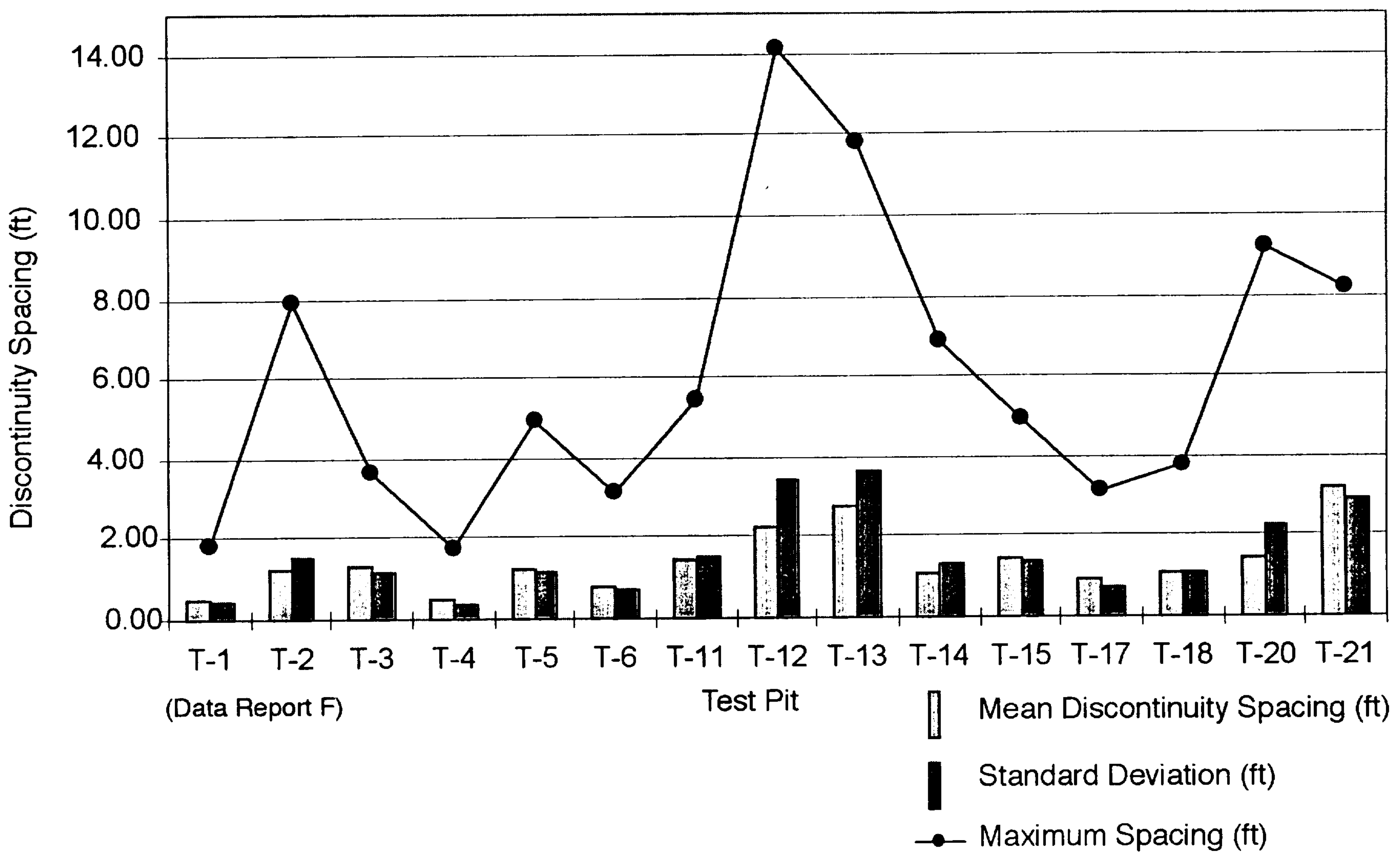
Rock Mass Discontinuities

Type	Typical Spacing	Lateral Continuity	Surface Roughness	Degree of Bonding	Dip Angle	Shear Strength
Bedding	0.5 to tens of feet	Tens to hundreds of feet	Moderately smooth to rough	Moderate to High	Gentle	Moderate to High
Joints	0.5 to 3.5 feet	Several to tens of feet	Smooth to rough	Low to Moderate	Moderately steep to steep	Moderate to Low
Faults	Tens to hundreds of feet	Tens to hundreds of feet	Smooth to rough	Low to Moderate	Steep	Low



Limit of Topo
Reservoir
Raw Water
cutslope
ISFSI
fillslope
T-21
N=15
T-20
N=25
01-E
01-G
N=13
T-19
T-13
N=23
T-4
N=52
T-5
N=49
T-3
T-2
N=86
T-12
N=16
01-H
N=21
01-F
01-D
N=25
T-1
T-11
N=36
01-C
N=38
DS-1
T-17
N=114
T-18
N=54
01-B
N=16
01-A
N=34
T-6
area


Termination
of Joint



Discontinuity Spacing (ft)
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
T-1
T-2
T-3
T-4
T-5
T-6
T-11
T-12
T-13
T-14
T-15
T-17
T-18
T-20
T-21
(Data Report F)
Test Pit
Mean Discontinuity Spacing (ft)
Standard Deviation (ft)
Maximum Spacing (ft)

Approach

- Kinematic Analyses:
  - Identify potential failure modes
  - Select model for pseudostatic analyses
- Pseudostatic Analyses:
  - Deterministic Factor of Safety
  - Evaluate Sensitivity of Input Parameters
  - Determine Anchor Force Requirements
- Evaluate Mitigation Options

Assumptions For Analyses

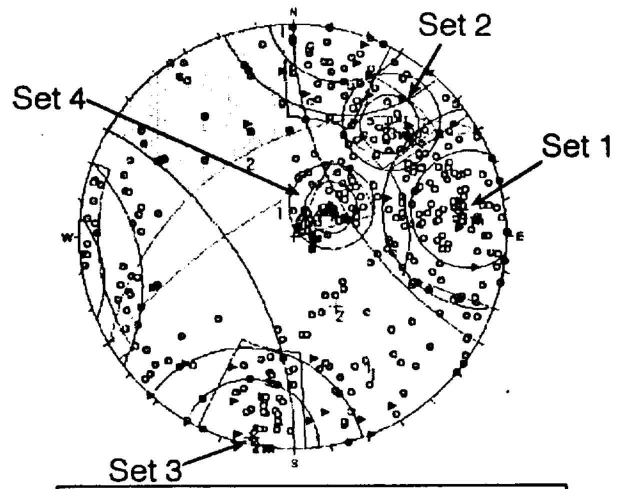
- Stability controlled by rock mass discontinuities
- Discontinuity shear strength is represented by the frictional component of the median Barton shear strength envelope
- We assume no benefit from cohesion
- Groundwater/rainwater collects in discontinuities up to half-height of the wedge
- Wedges are limited within the outermost 20 to 25 feet of slope
- Tension cracks exist at the top of the cutslope

Software

- Qualified software
- Kinematic Analyses – DIPS Version 5.041 (Rocscience, 1999)
- Pseudostatic Analyses – SWEDGE Version 3.06 (Rocscience, 1999)

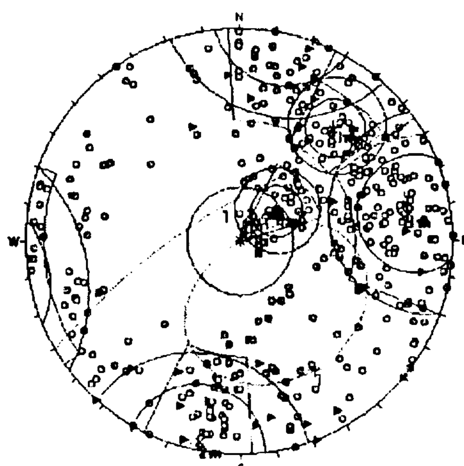
Kinematic Analyses
Input Parameters

- Geologic mapping data
- Discontinuity surveys in trenches and cuts
  - Bedrock bedding
  - Joints
  - Faults

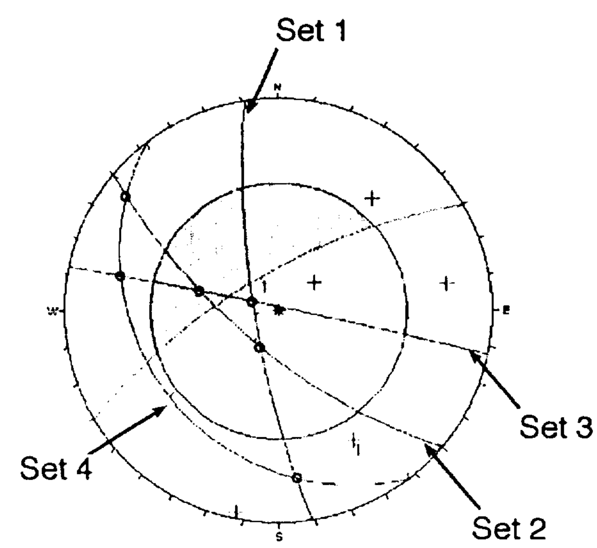


A. Topple hazard (low hazard)

Southern Hemisphere Projections



B. Planar sliding hazard (low to moderate hazard)



C. Wedge sliding hazard (high hazard)

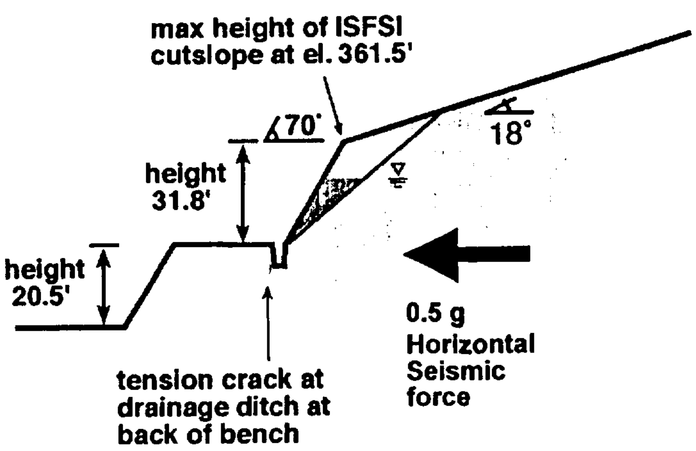
(SAR, PG&E, 2001)

Results - Kinematic Analyses

Cutslope	Topple	Planar Sliding	Wedge Sliding	Pseudostatic Analyses	Mitigation Required?
Eastcut	Low	Mod. To High	Mod. To High (3 sets)	Yes	Yes
Backcut	Low	Low to Mod.	High (4 sets)	Yes	Yes
Westcut	High	Low	Very Low	No	No

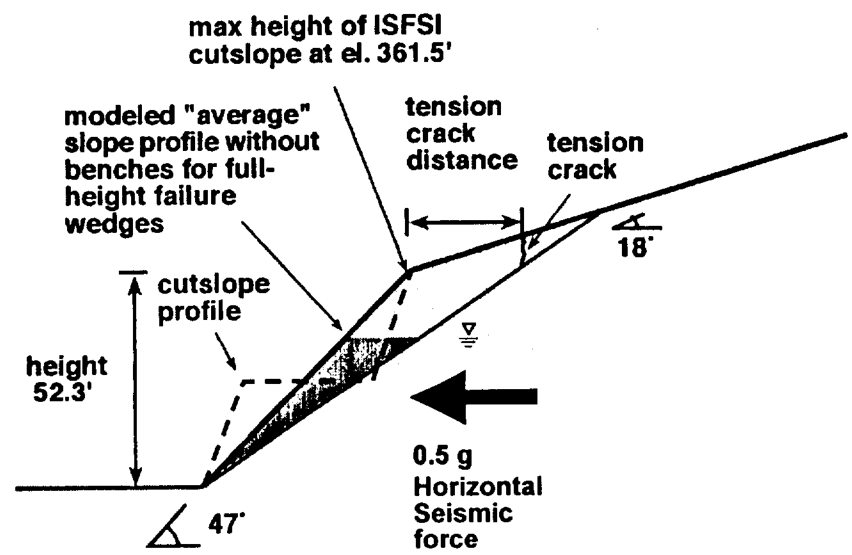
Pseudostatic Analyses Input Parameters

- Barton mean shear strength values
- Laboratory direct shear test results
- Seismic loading of 0.5g acting as a uniform horizontal force
- Cutslope geometry from design drawings
- Wedge intersections from kinematic analyses
- Variable wedge geometry, shear strength



Single bench wedge

SWEDGE analysis



Total cut-height wedge as modeled by SWEDGE program

(From SAR, Fig.2.6-60)

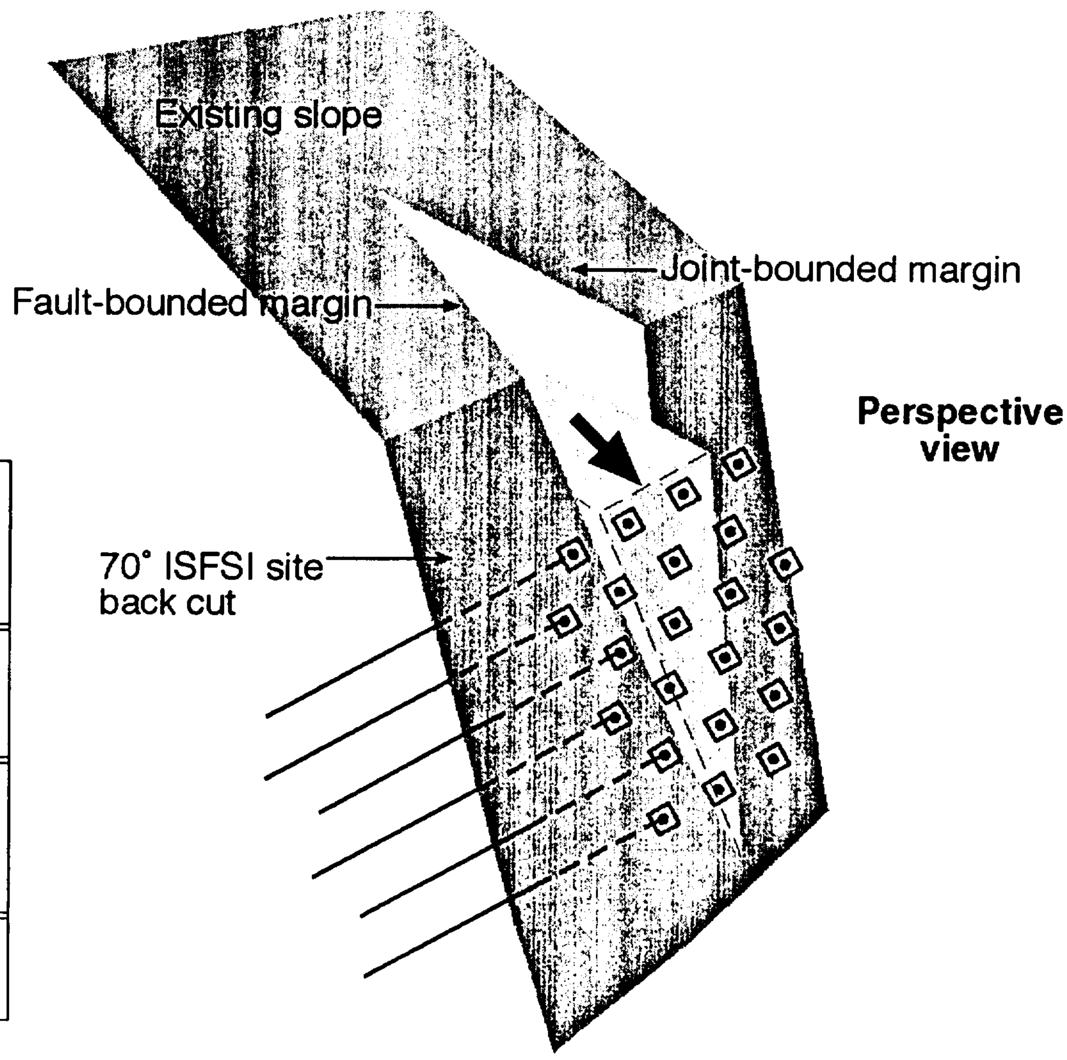
Dynamic Factor of Safety = 1.3
Capacity 34 kips
Length 23 feet + Bond

Critical wedges

Backcut critical wedge weight	**Dynamic factor of safety** without support	**Dynamic factor of safety** with support	**Anchor characteristics** per anchor capacity*	**Anchor characteristics** anchor length
1784 kips	0.62	1.3	33.9 kips	13 feet
4475 kips	0.63	1.3	32.1 kips	23 feet
40 kips	0.0	1.4	18.6 kips	7 feet
10 kips	0.3	1.7	9.4 kips	4 feet

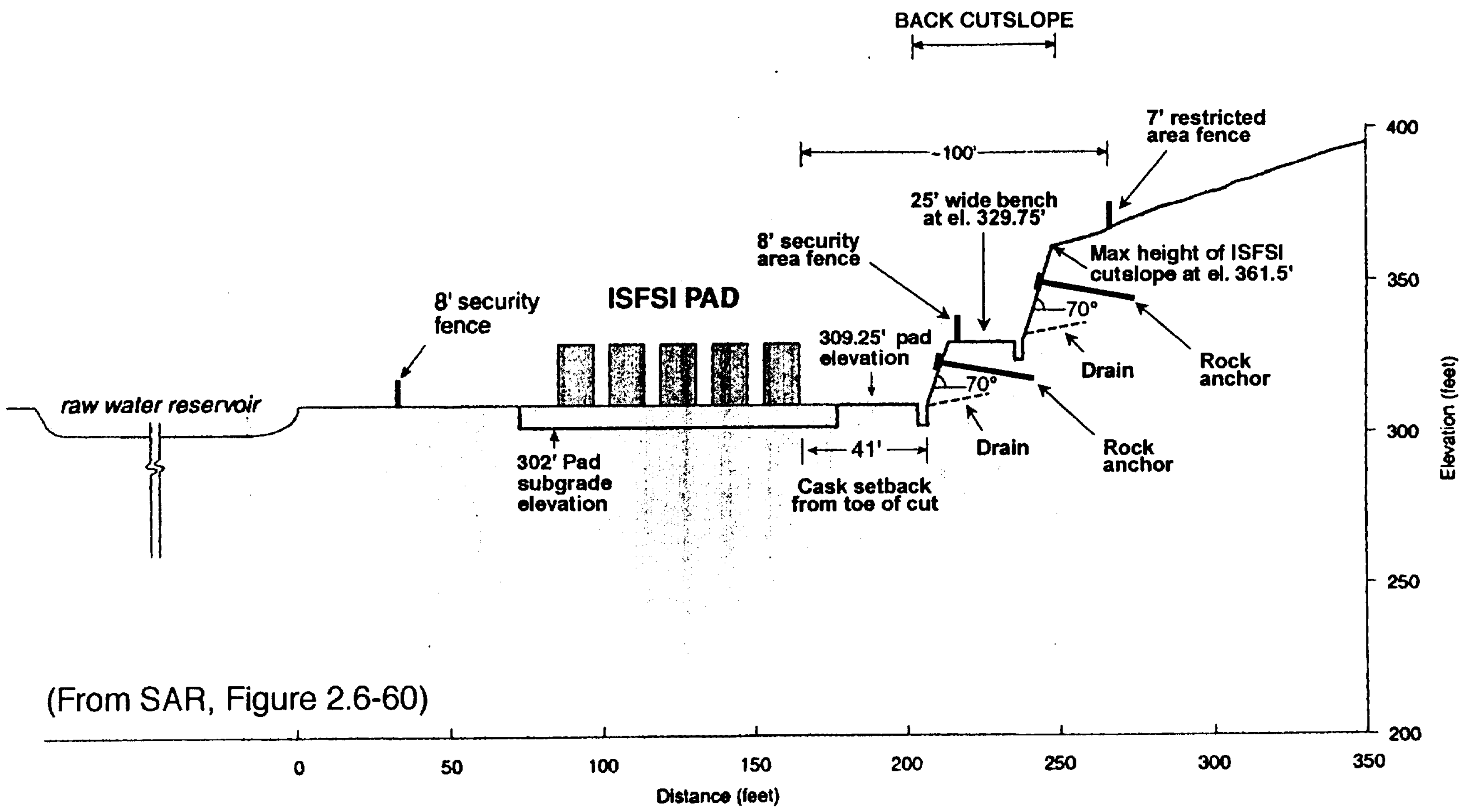
Eastcut critical wedge weight	**Dynamic factor of safety** without support	**Dynamic factor of safety** with support	**Anchor characteristics** per bolt capacity	**Anchor characteristics** anchor length
34 kips	0.54	1.3	9.0 kips	3.5 feet
23.8 kips	0.0	1.4	8.4 kips	3.5 feet

* 5' x 5' pattern



Mitigation

- Rock anchor
- Drainage
- Debris fences
- Shotcrete
- Setback



(From SAR, Figure 2.6-60)

Conclusions – Cutslope Geology

- Proposed ISFSI cutslopes will be excavated in dolomite, sandstone, and friable rock
- Rock mass discontinuities control cutslope stability
- Discontinuity spacing limits size of potential blocks to less than 20 to 25 feet

Conclusions – Cutslope Stability

- Stability analyses shows that cutslopes exhibit high likelihood for wedge failure
- Rock anchors will effectively stabilize cutslopes to achieve a dynamic, saturated Factor of Safety of 1.3
  - 34 kip anchors at 5' X 5' spacing
  - Penetration lengths of up to 25'
  - Proposed mitigation measures provide high margin of safety

Conclusions – Cutslope Stability

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NRC/PG&E Open Meeting, San Francisco, CA
Diablo Canyon Independent Spent Fuel Storage Installation

Response to NRC question

William D. Page
Senior Engineering Geologist
PG&E Geosciences Department

PG&E

April 11, 2002

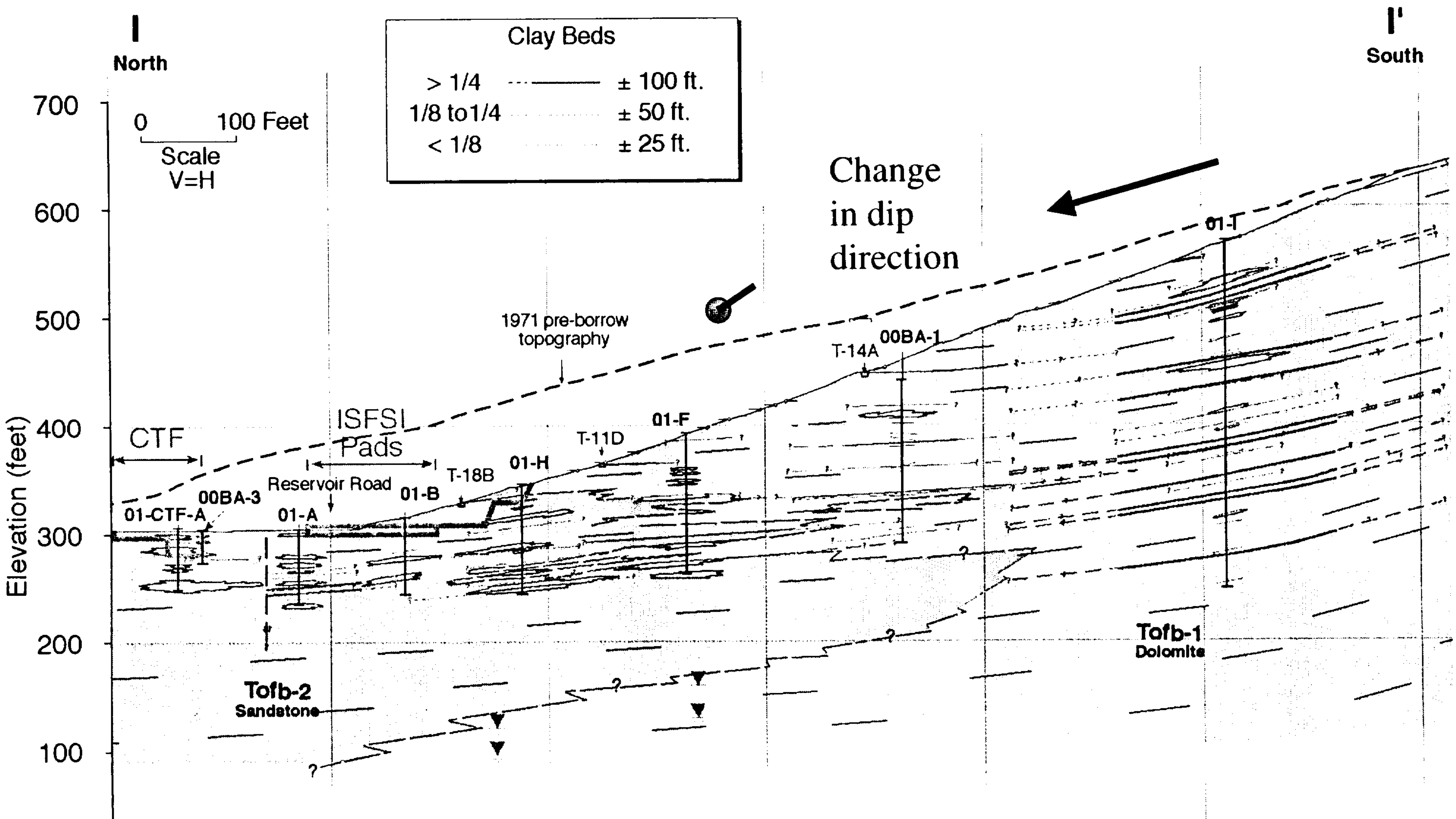
Question: Explain Degree of Confidence in Results

- Input parameters used for modeling potential large-scale rock mass movements are realistic and conservative
- Confidence in predicted foundation conditions at CTF, ISFSI Pads and ISFSI cutslopes

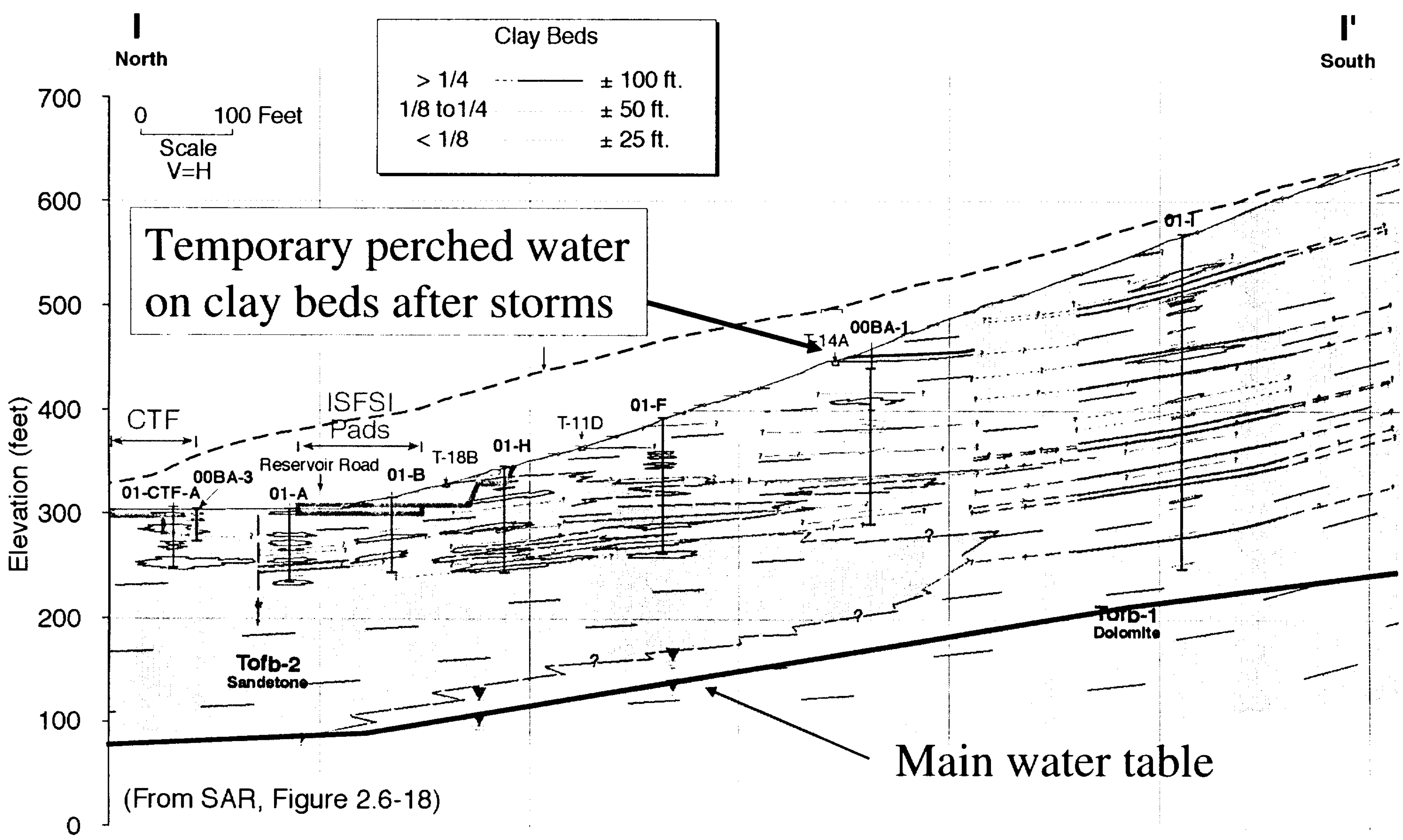
Input Parameters for Modeling

- Geometry of clay beds well understood
- Groundwater conditions known, clay beds assumed saturated

Dip Direction



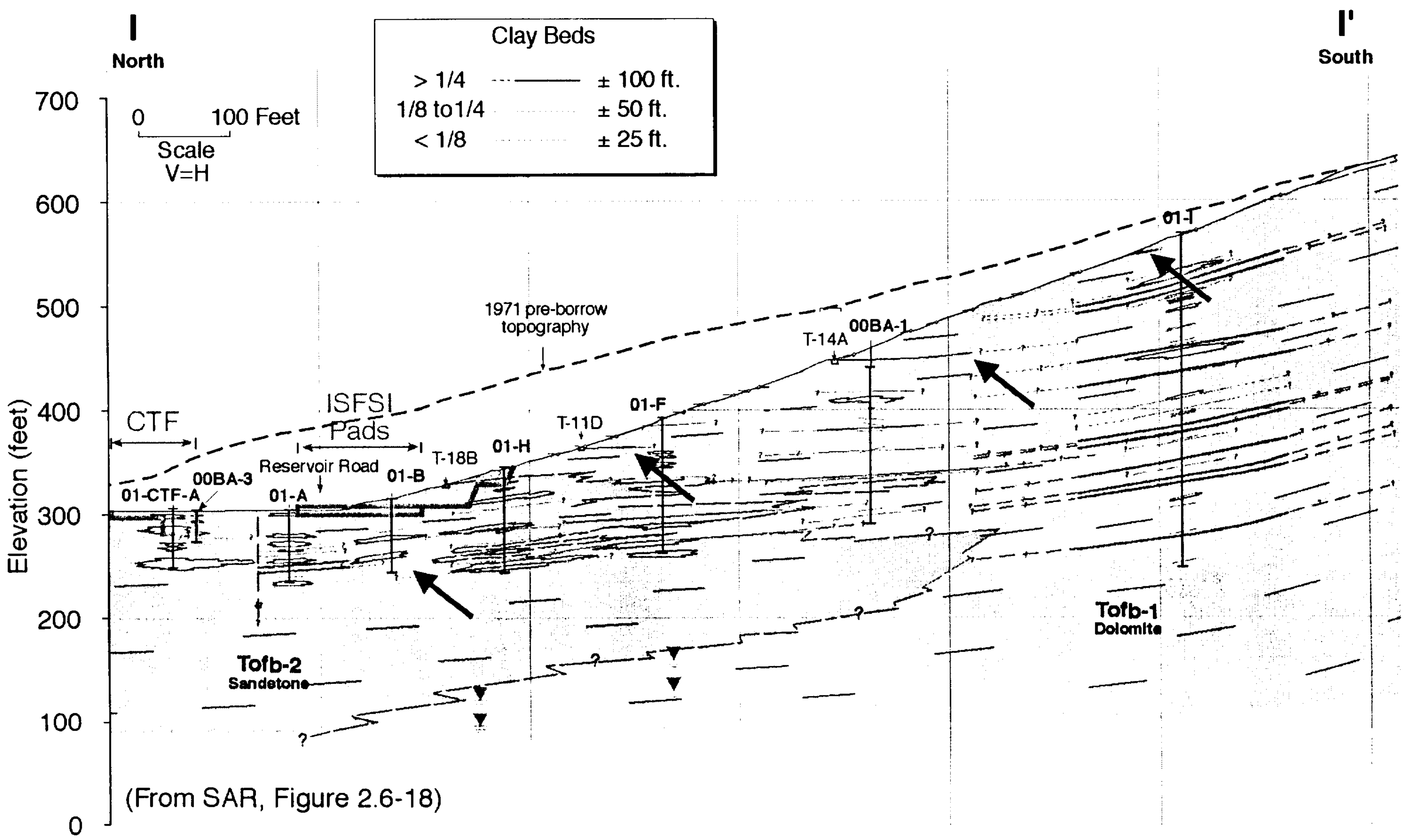
(From SAR, Figure 2.6-18)


I
North
I'
South
Clay Beds
> 1/4 ± 100 ft.
1/8 to1/4 ± 50 ft.
< 1/8 ± 25 ft.
0 100 Feet
Scale
V=H
Temporary perched water
on clay beds after storms
Elevation (feet)
700
600
500
400
300
200
100
0
CTF
ISFSI
Pads
Reservoir Road
01-CTF-A
00BA-3
01-A
01-B
T-18B
01-H
T-11D
01-F
T-14A
00BA-1
01-T
Tofb-2
Sandstone
Tofb-1
Dolomite
Main water table
(From SAR, Figure 2.6-18)

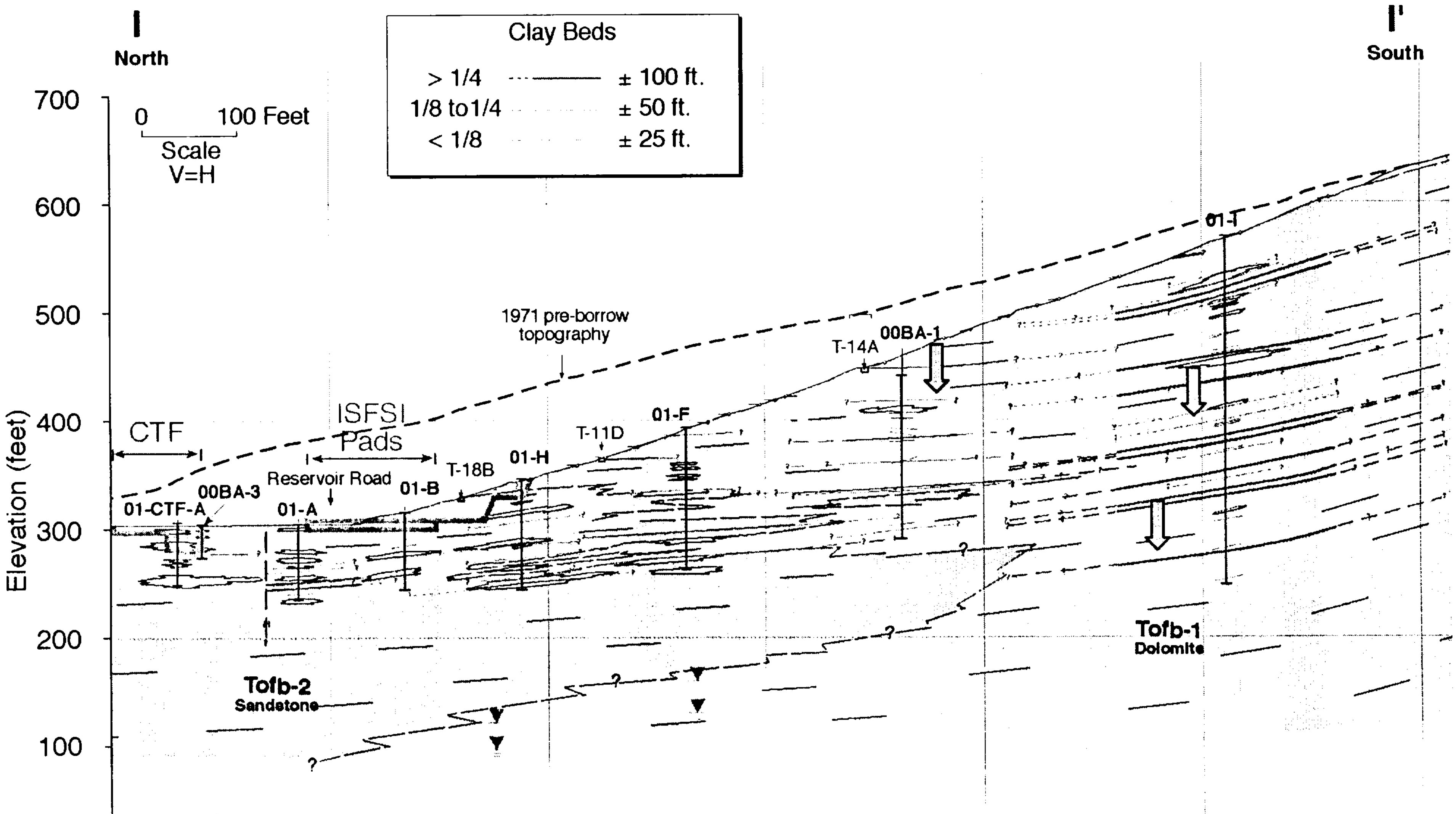
Large-scale Mass Movements

- Geologic interpretations of extent of clay beds is conservative, but not extreme
- Potential slide planes are chosen to follow the full extent of more extensive clay beds and step between clay beds, this assumes minimum rupture of rock
- Rock to rock contact along potential slide plane along clay beds not factored into model, this would increase the clay strength from that used

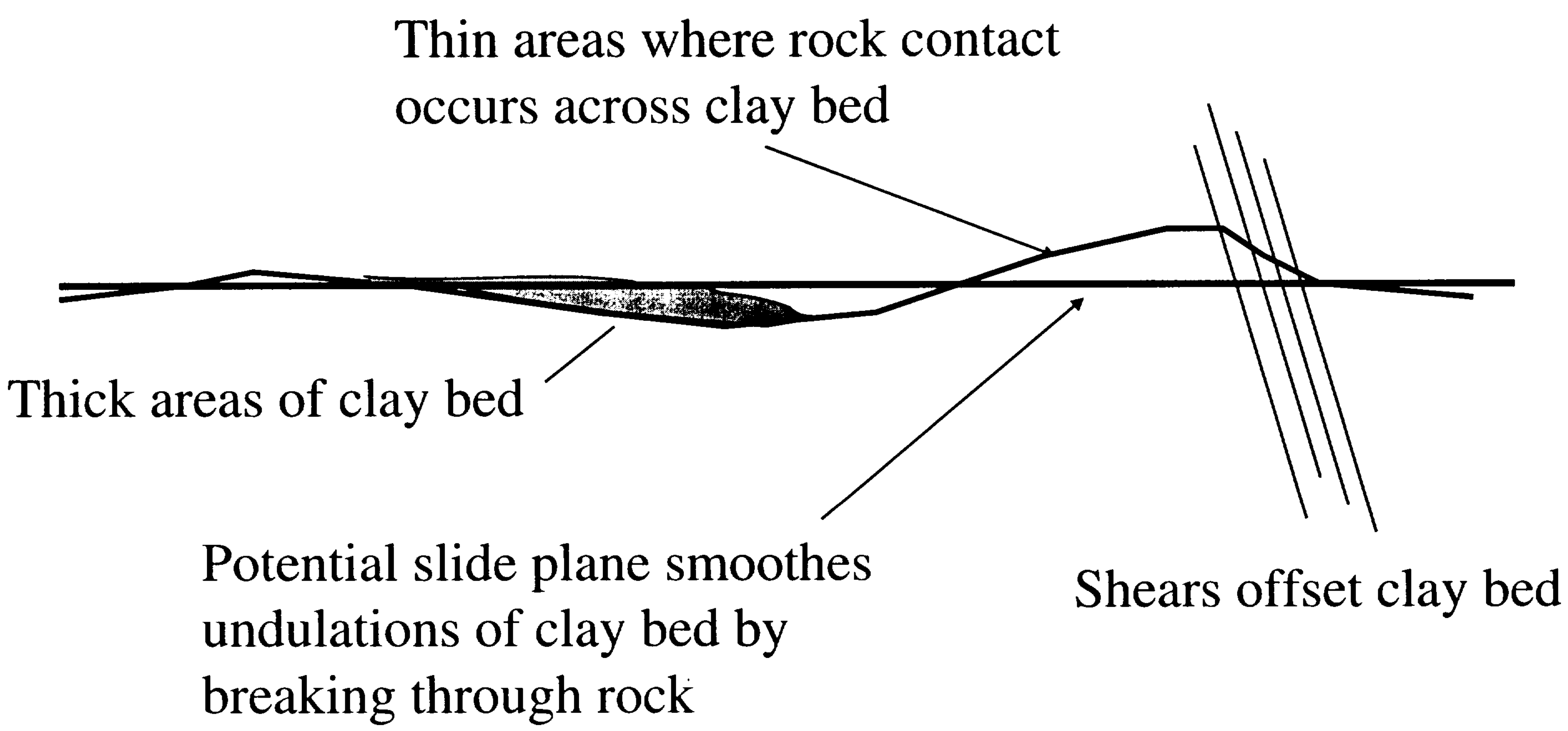
Clay Beds Not Continuous



Clay Bed Extent Based on Thickness



(From SAR, Figure 2.6-18)



Potential Slide Plane Breaks through
Rock along Clay Bed

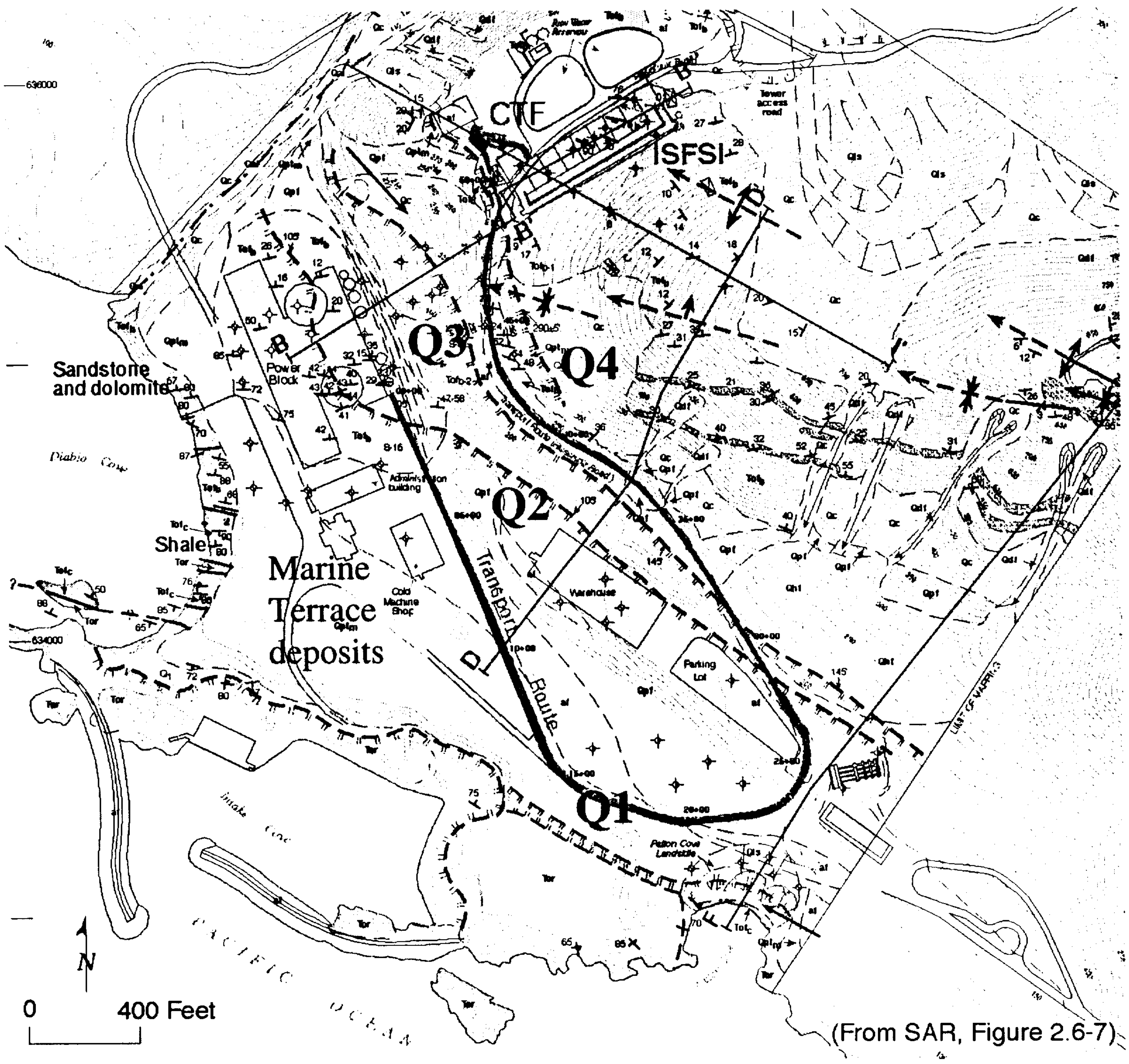
Evidence of No Landslides at ISFSI

- No evidence on pre-1971 air photos
- No evidence in studies for and excavation of borrow site
- No evidence of any fissures or fissure fills in trenches for ISFSI

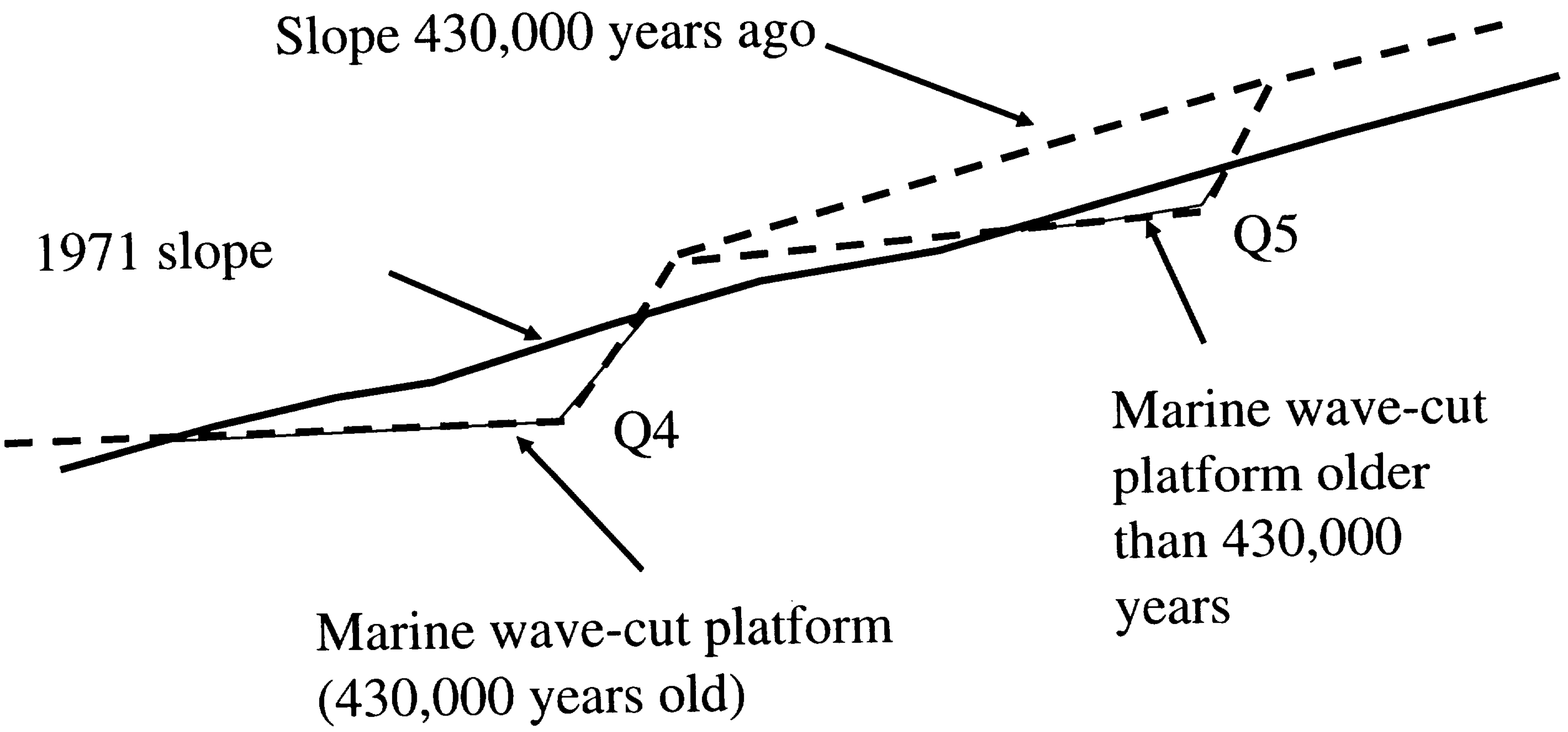
Assumed Displacement of Large Scale Slide Mass

- Fractures in the slope larger than 3 to 4 inches would have left a record on the slope
  - No vegetation lineaments (similar to the zones of intense growth in filled trenches)
  - No open fractures or soil–filled fractures in trenches on slope
- Hillslope is 430,000 years old
- Subjected to many large earthquakes
  - Assumed 4 inches would occur in one slide event

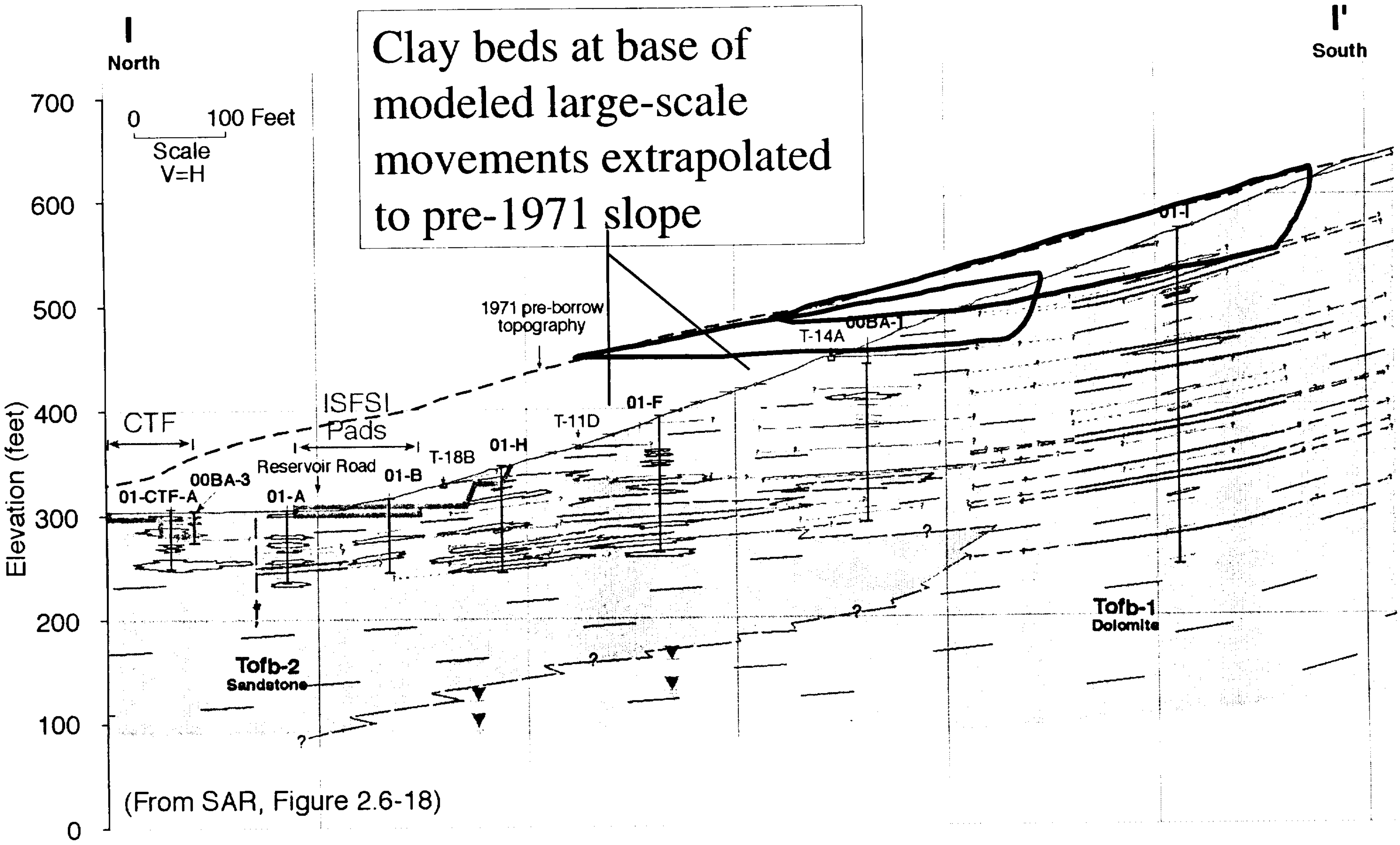
Marine Terraces



(From SAR, Figure 2.6-7)

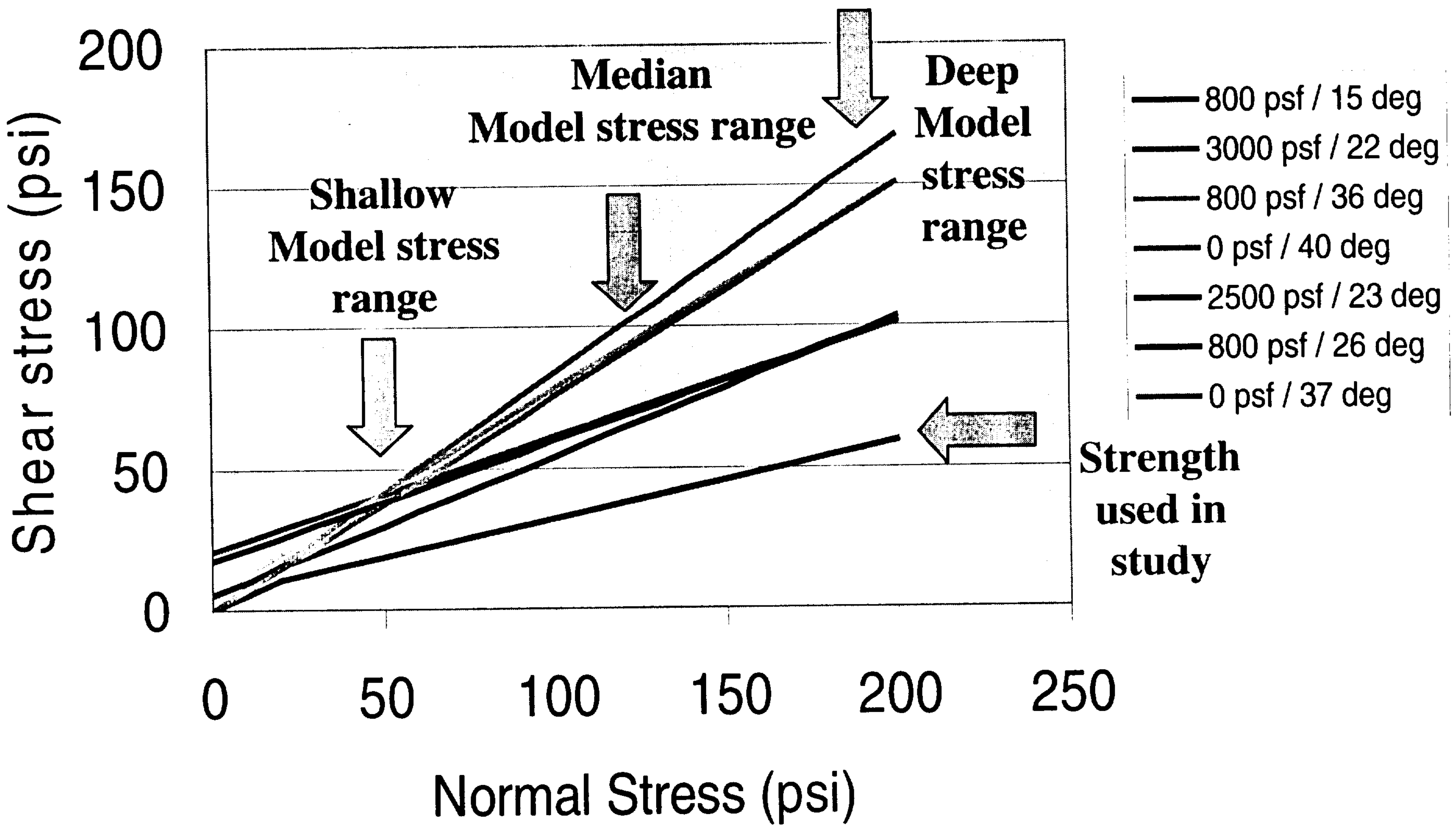


Hill Slope is 430,000 Years Old, but degraded a Few Tens of Feet



I
North
I'
South
Clay beds at base of modeled large-scale movements extrapolated to pre-1971 slope
0
100 Feet
Scale
V=H
700
600
500
400
300
200
100
0
Elevation (feet)
1971 pre-borrow topography
CTF
ISFSI Pads
Reservoir Road
01-CTF-A
00BA-3
01-A
01-B
T-18B
01-H
T-11D
01-F
T-14A
00BA-1
01-I
Tofb-2
Sandstone
Tofb-1
Dolomite
(From SAR, Figure 2.6-18)

Results of Sensitivity Study
Clay Bed Strengths



Confidence in Predicted Foundation Conditions at CTF, ISFSI Pads and Cutslopes

- High confidence in rock types predicted
  - Sandstone
  - Dolomite
  - Friable Sandstone
  - Friable Dolomite
  - Clay beds

Interpretations with Less Certainty

- Locations and percentage of rock types not known with certainty
- Friable diabase may be encountered and is expected to have the same properties as friable sandstone
- Attitude of clay beds uncertain, more clay beds may be exposed
- Precise location of faults uncertain, other shear zones are expected

Conclusion

- High degree of confidence that there will be no significant surprises
- Features will be mapped during construction
- Planned mitigation measures will be applied as appropriate

April 9, 2002

Diablo Canyon ISFSI Site Tour

PG&E

Attachment 3

Diablo Canyon Dry Cask Tour Agenda
April 9, 2002

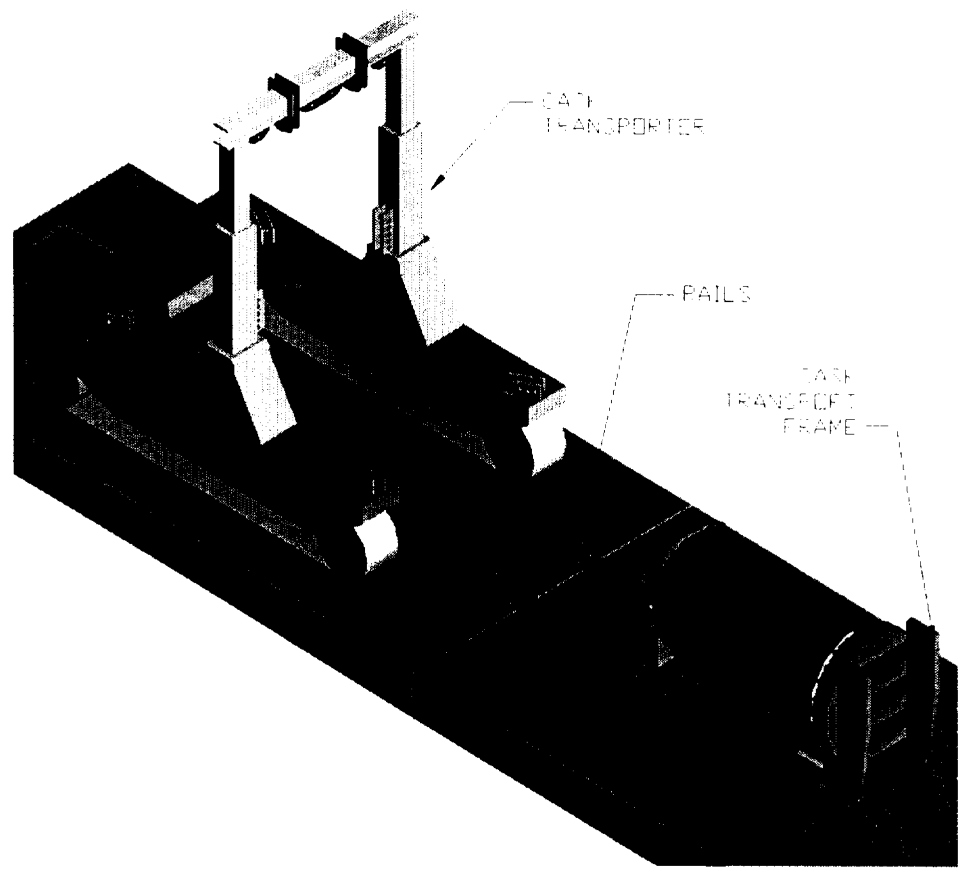
8:00 – 8:25	Maintenance Shop Building Training, Badging, Dosimetry
8:30 – 8:35	Canyon Room (Breakfast provided) Intro by Jearl Strickland, USFP Manager
8:35– 9:30	Canyon Room Part 50 and Part 72 Presentations
9:30 – 9:40	Break
9:40 – 10:40	Canyon Room Geotechnical Presentation

Diablo Canyon Dry Cask Tour Agenda
April 9, 2002

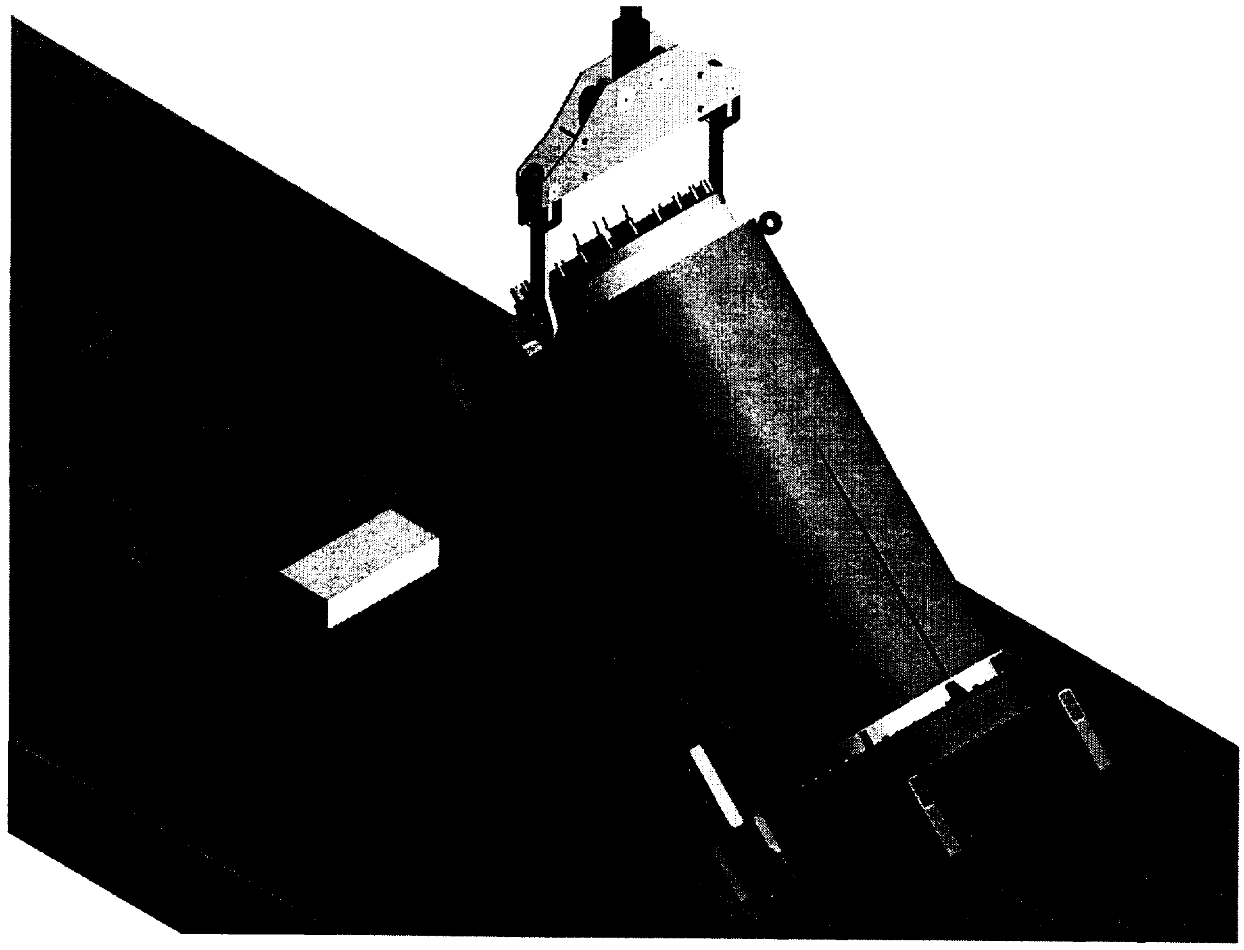
10:45 – 12:15	Board bus in front of Training Building Geosciences Tour
12:15 – 1:00	Training Building, Room 123 Lunch
1:05 – 4:00	Board bus in front of Training Building Field/ISFSI Site Inspection (Outside Protected Area)
4:00 – 5:00	Training Building, Room 123 Closure Activities, Q&A

Process for Loading Used Fuel into Dry Storage

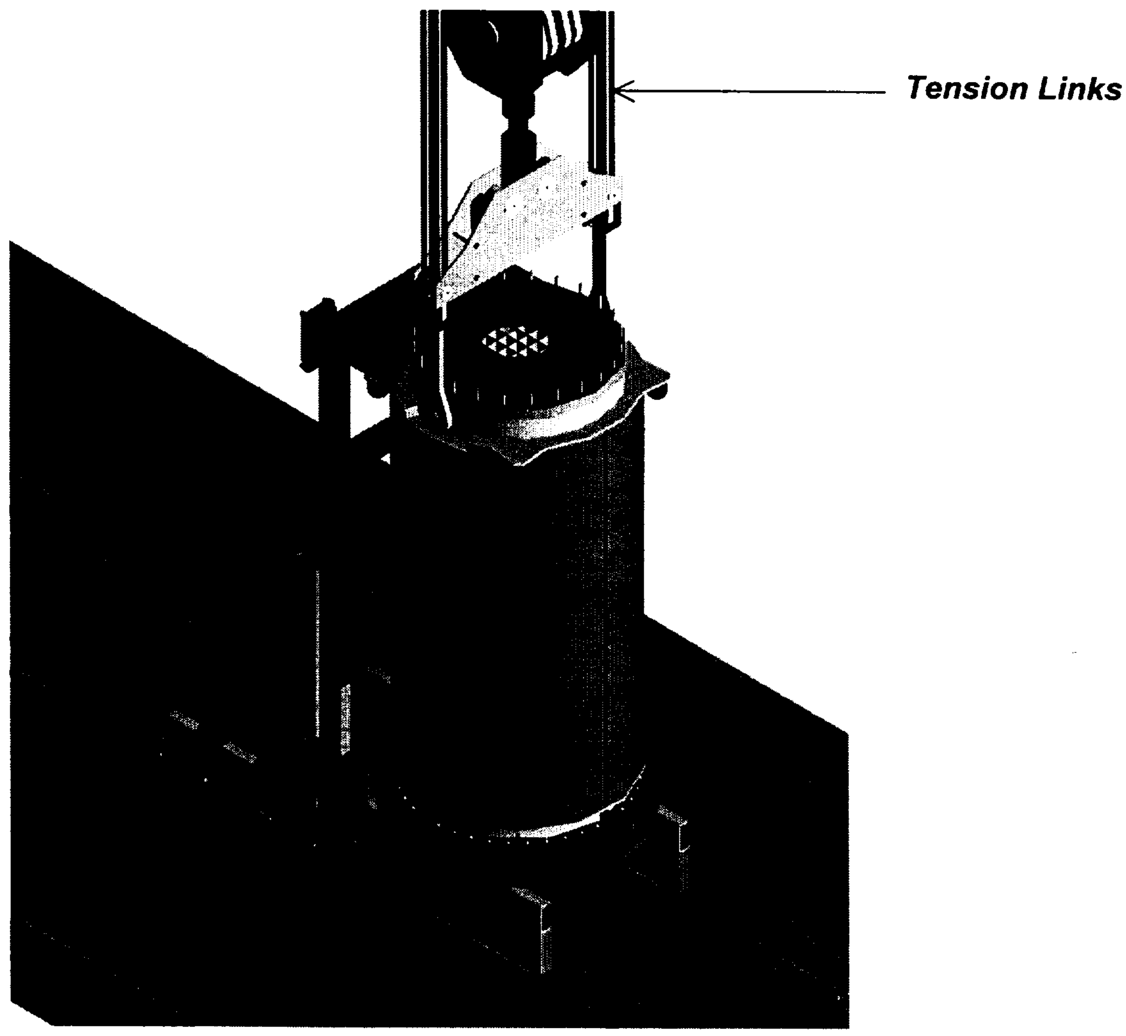
Transfer Cask Placement in Fuel Handling Bldg



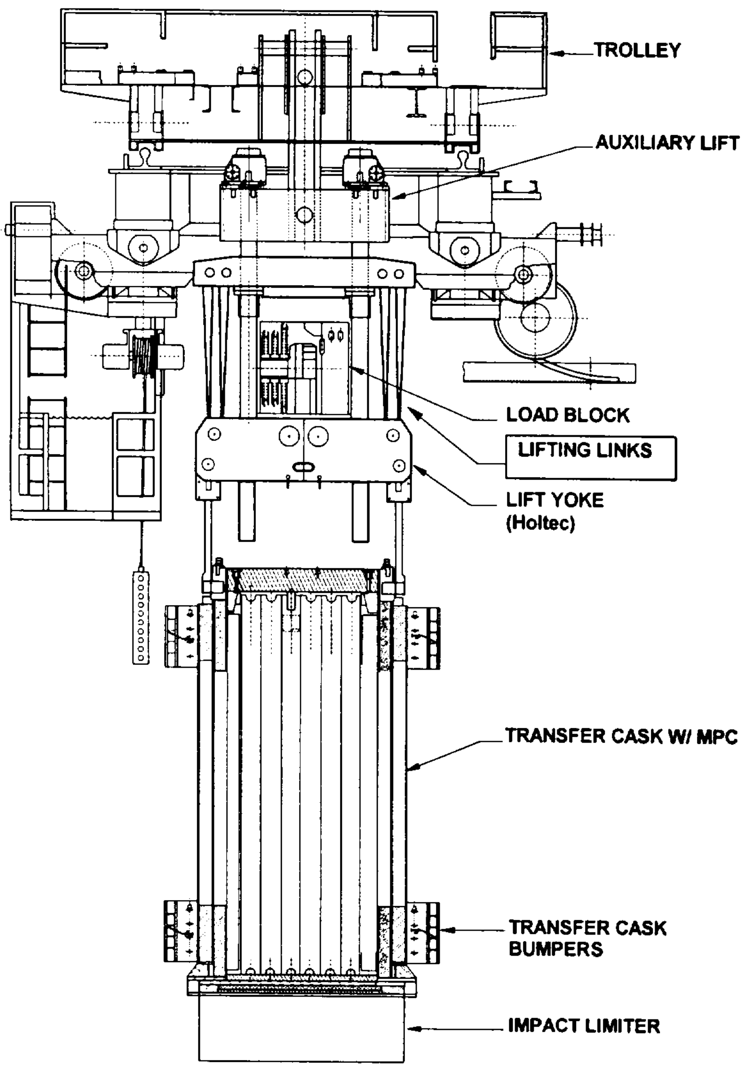
Transfer Cask Upending in Fuel Handling Bldg



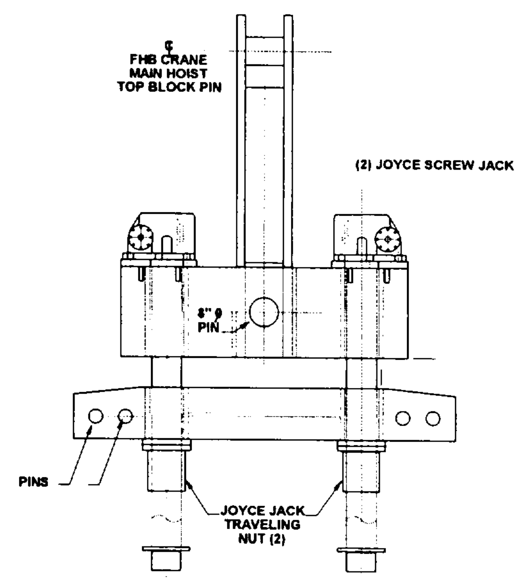
Cask Transport Frame Stabilizer



FUEL HANDLING BUILDING CRANE

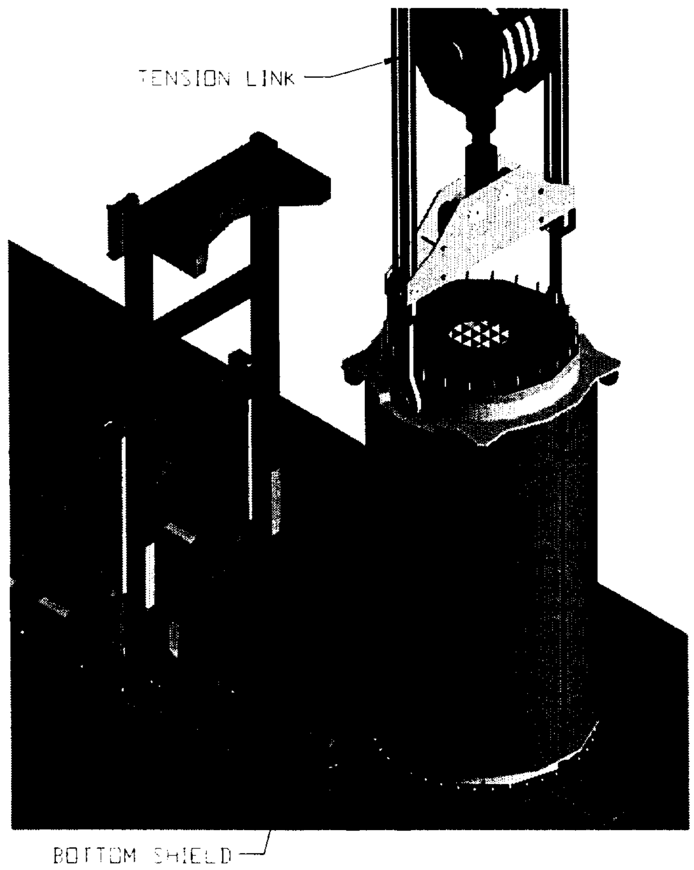


AUXILIARY LIFT

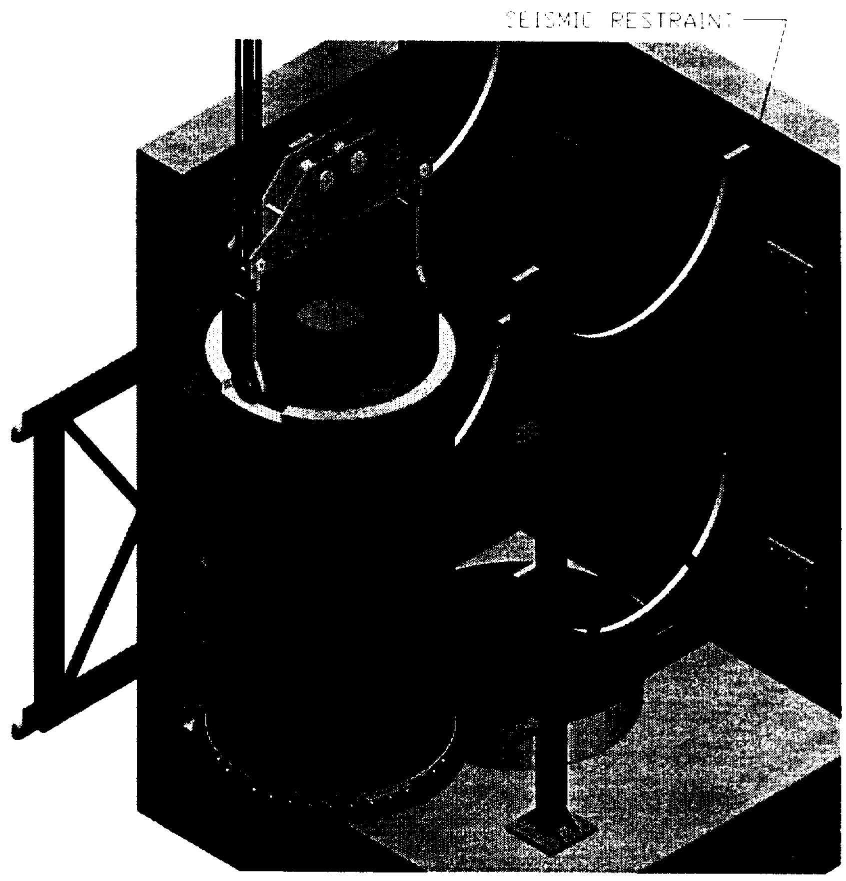


PG&E

Transfer Cask Removal From the Cask Transport Frame



Transfer Cask Placement in the Cask Washdown Area

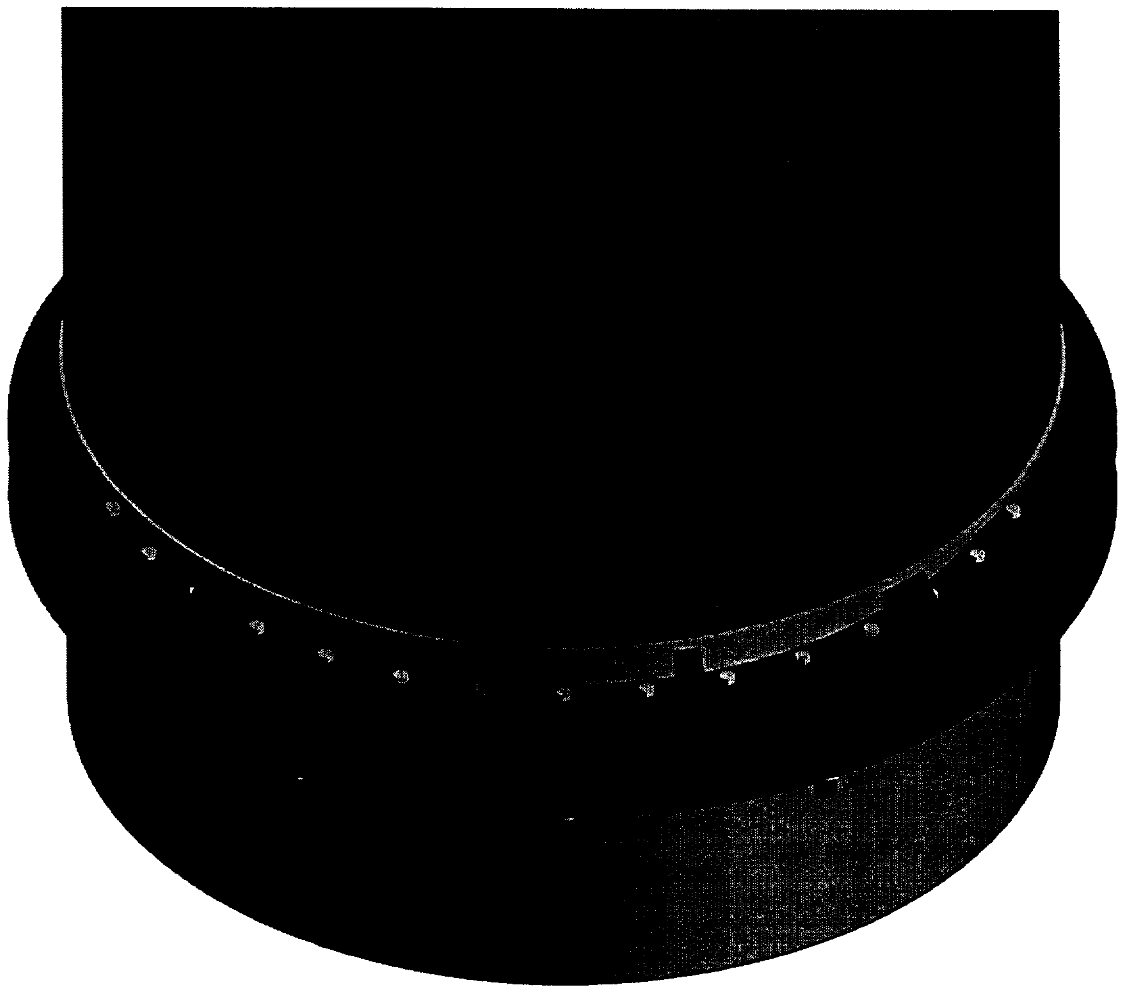


Note: Main hook not shown, but is engaged

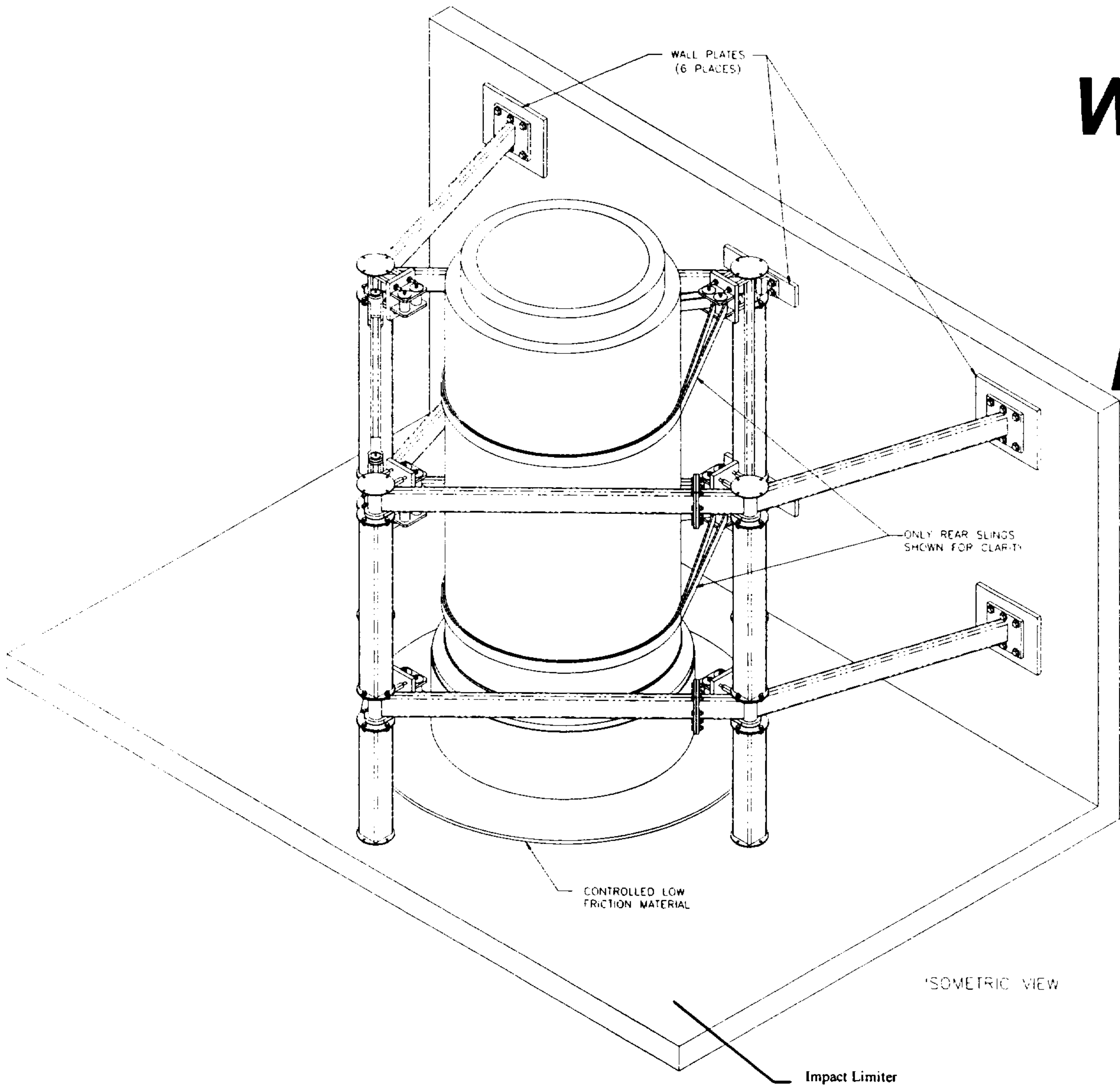
Note: Seismic Restraint is not current version in this slide

PG&E

Attachment of Impact Limiter

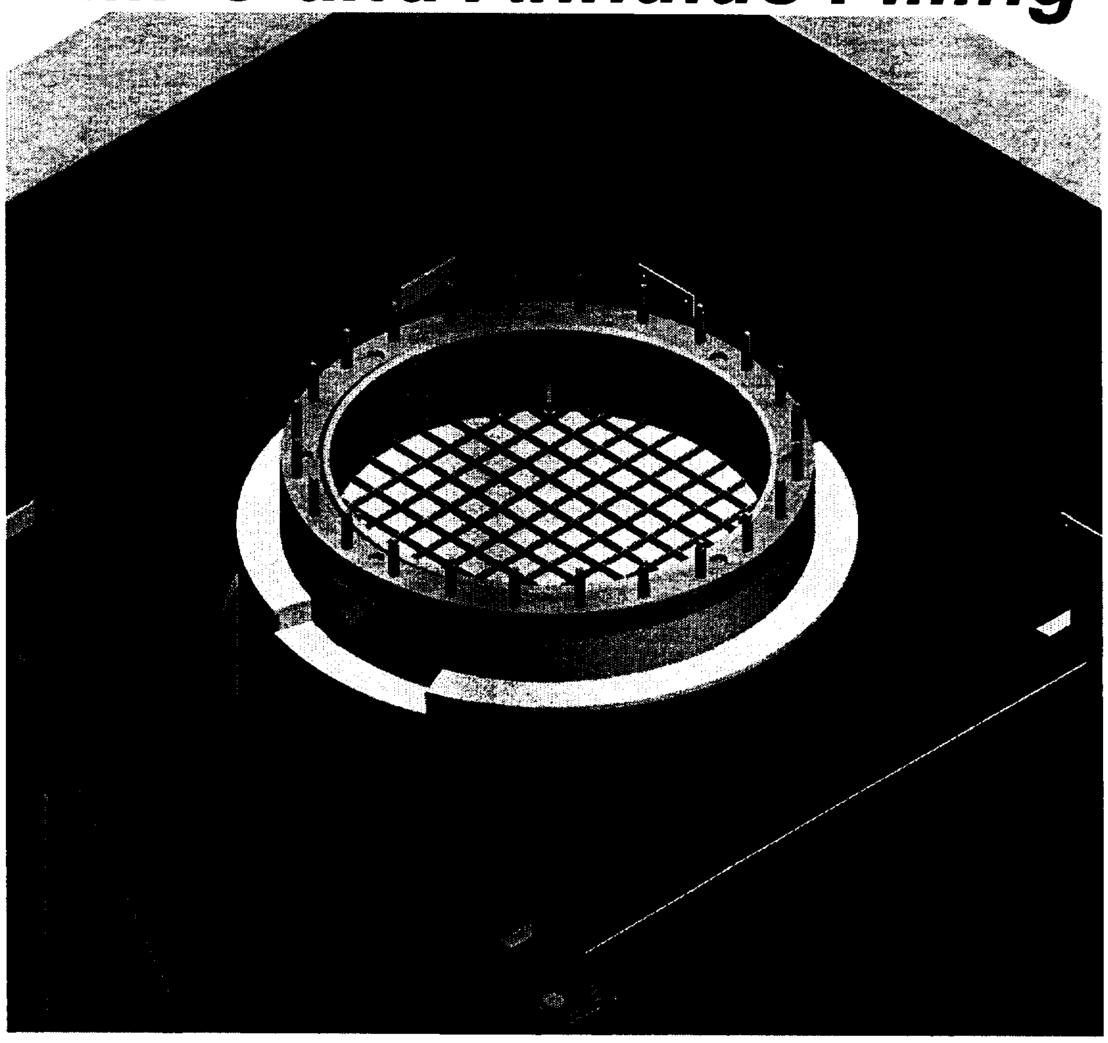


Cask Washdown Area Seismic Restraint



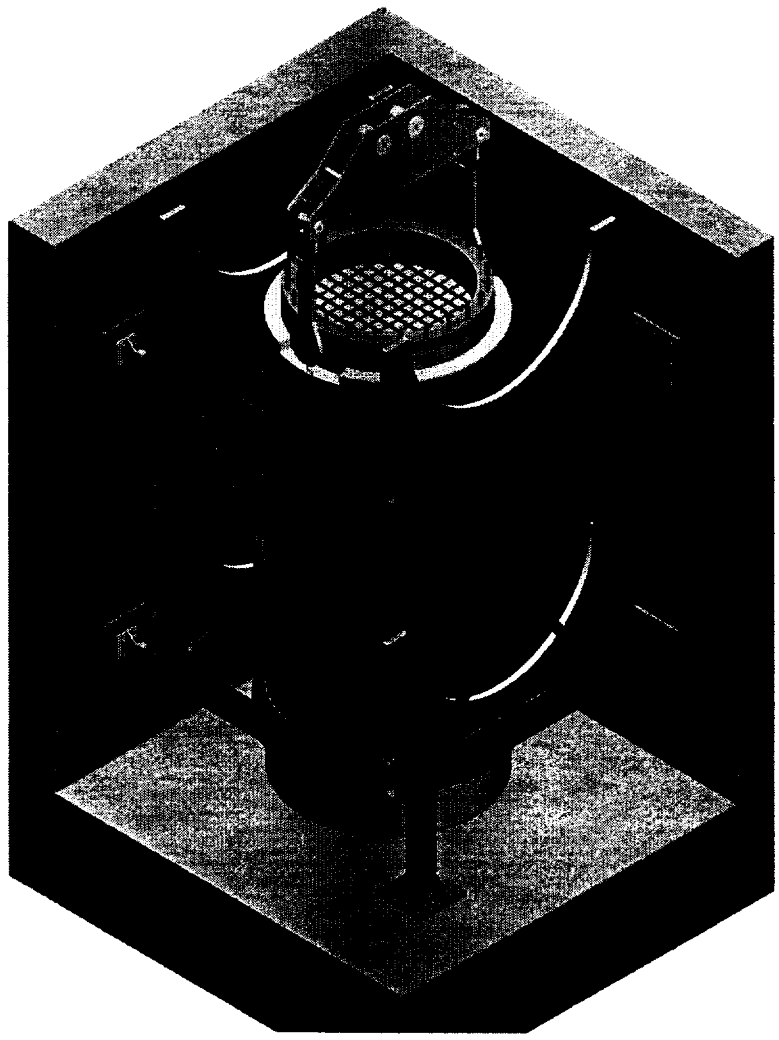
←

MPC and Annulus Filling



Note: Seismic restraint is not current version in this slide

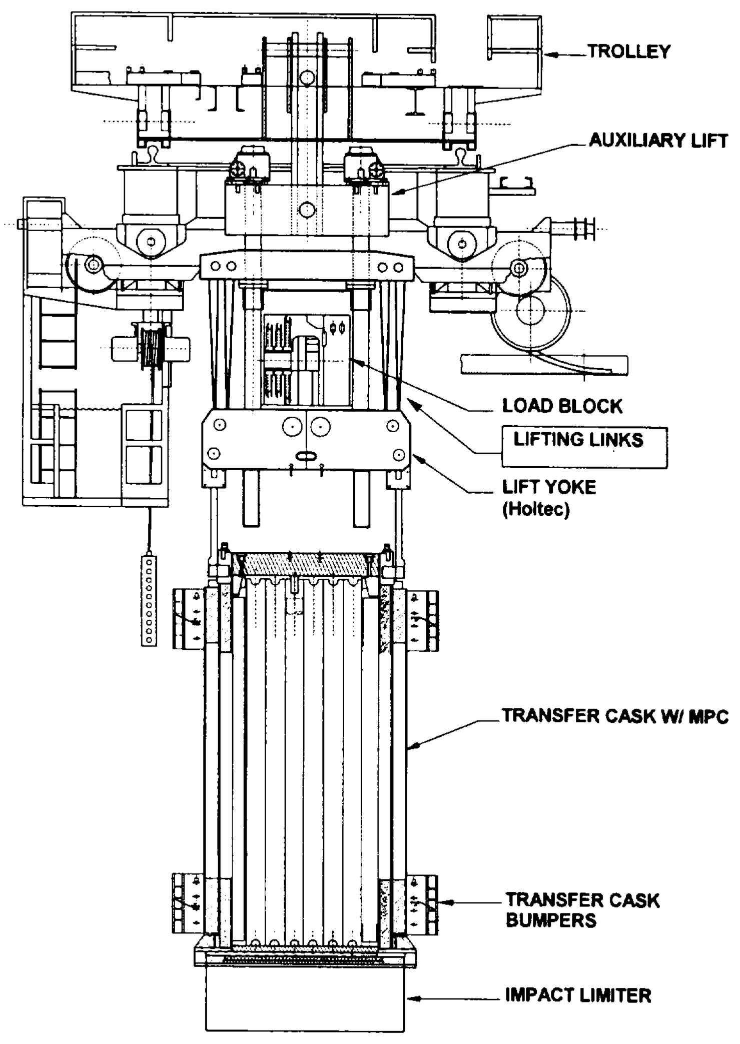
Transfer Cask Removal from Frame

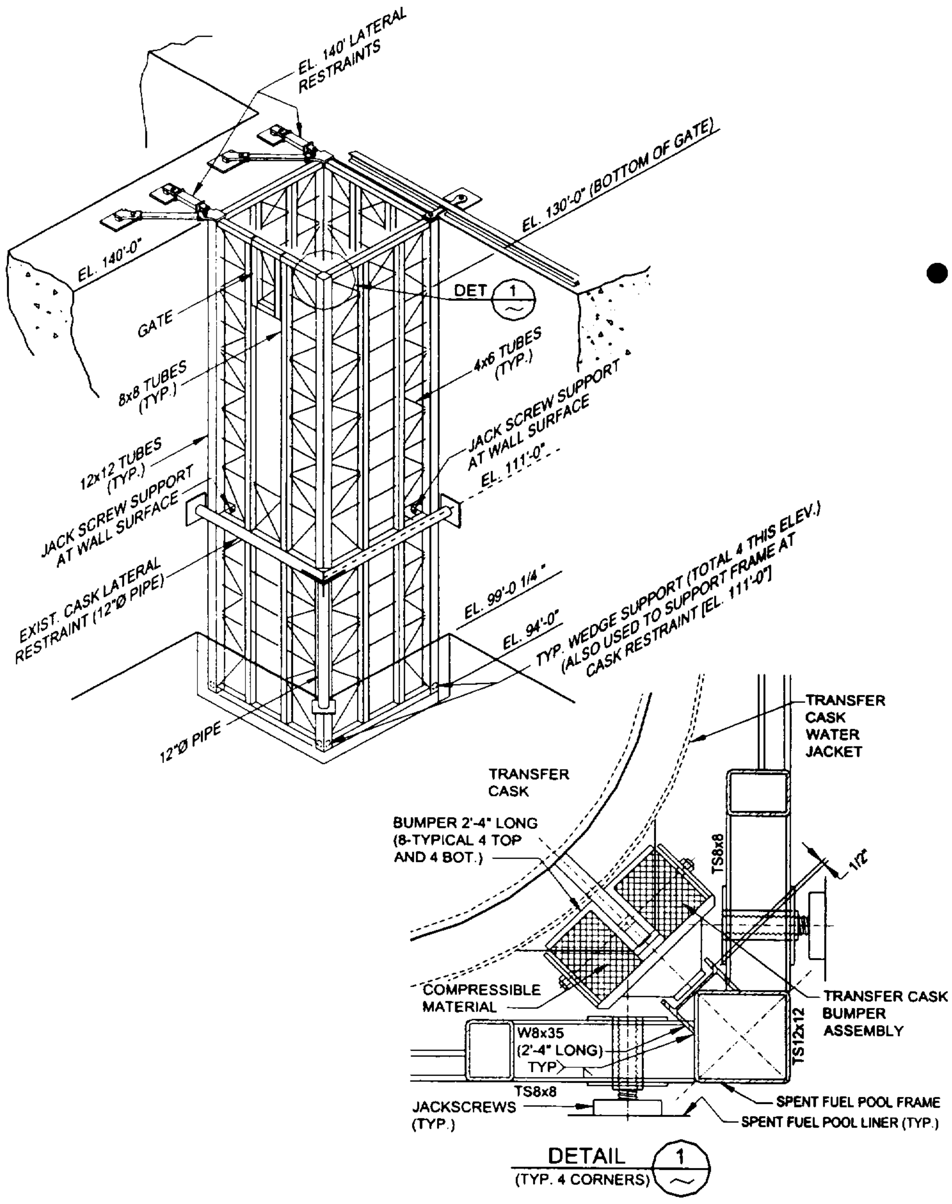


Note: Main hook is engaged, but not shown

Note: Seismic Restrain is not current version in this slide

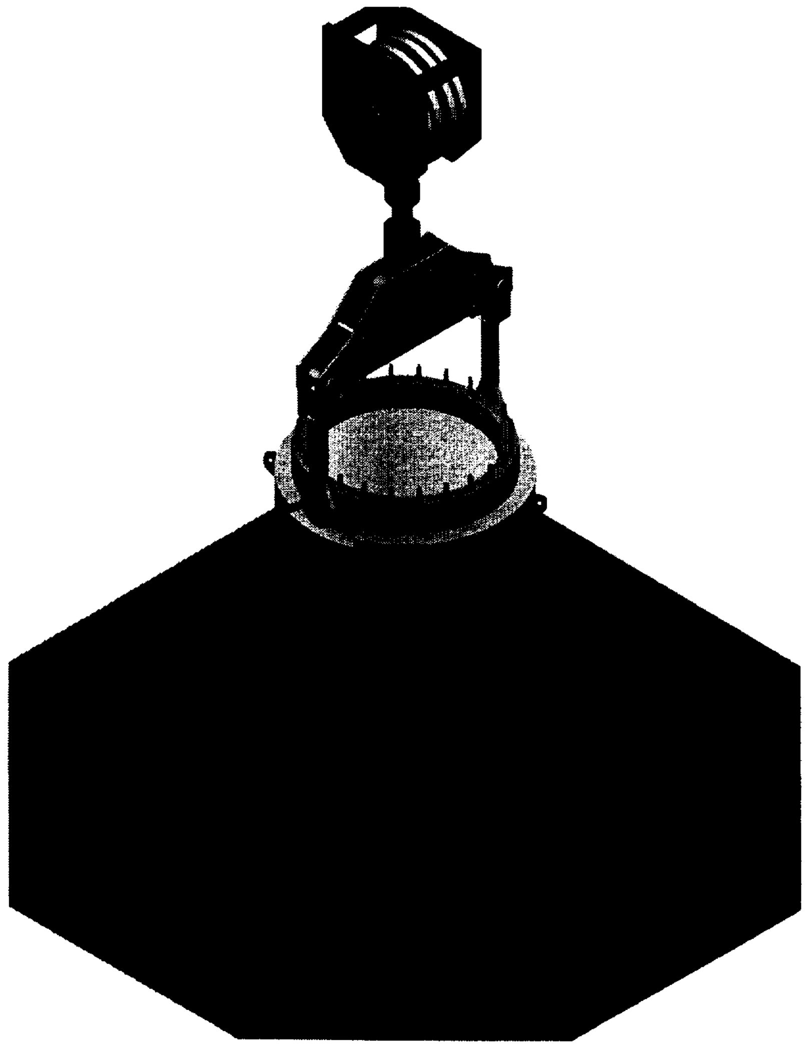
TRANSFER CASK READY FOR HORIZONTAL MOVEMENT TO SFP





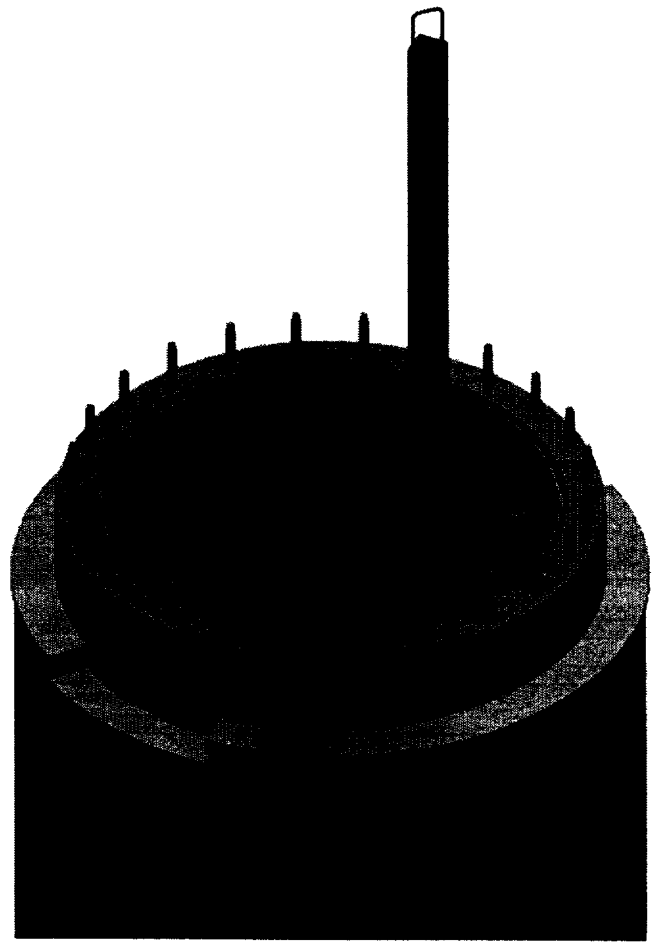
- SPENT FUEL POOL STRUCTURAL FRAME DETAIL

Transfer Cask Placement in Pool after Engaging in SFP Frame

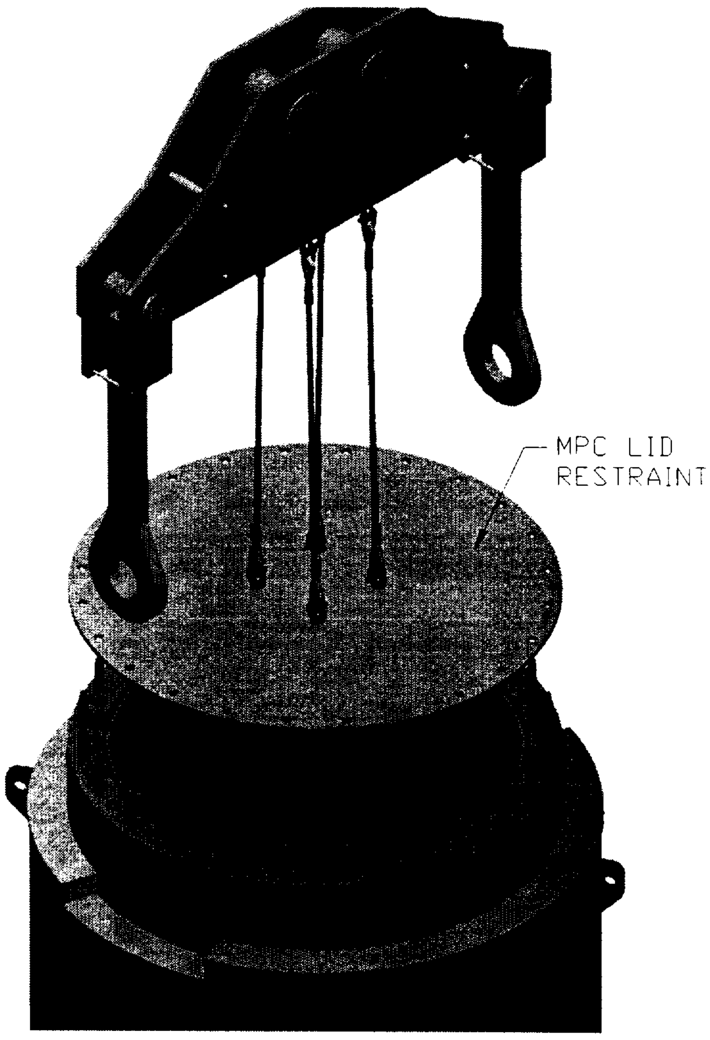


PG&E

Spent Fuel Loading into MPC

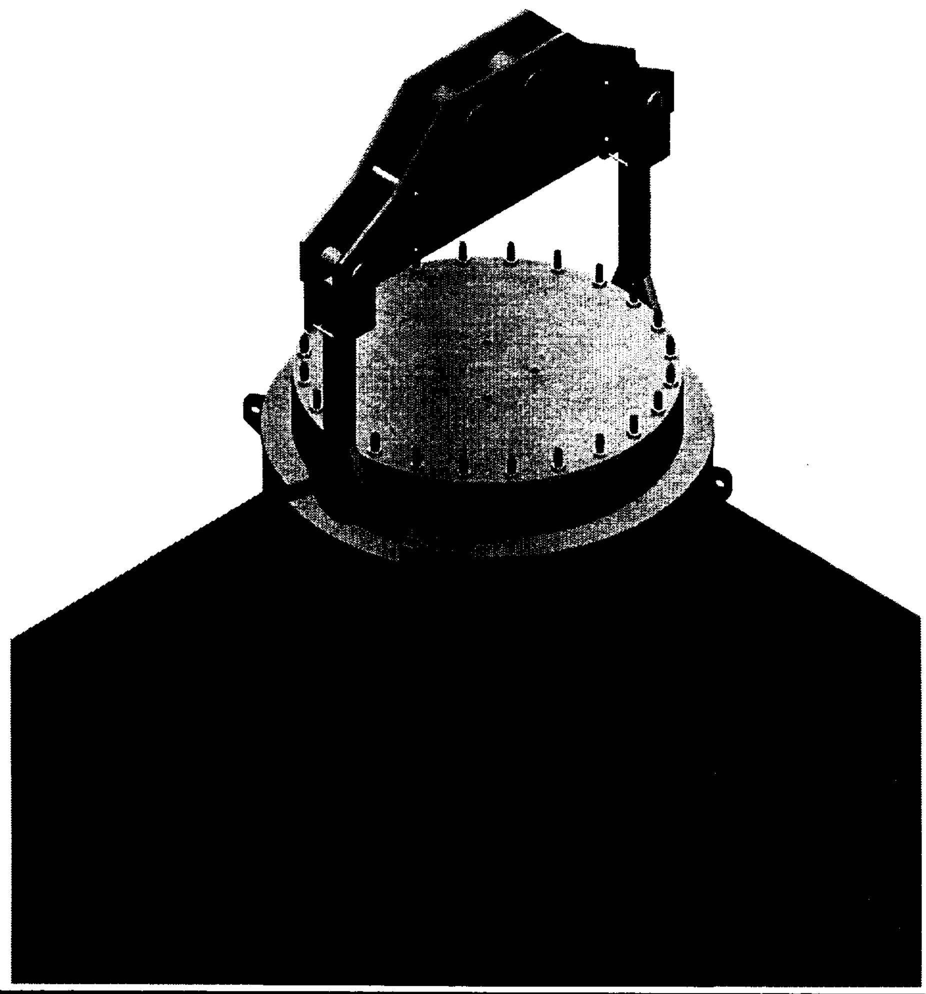


MPC Lid Installation



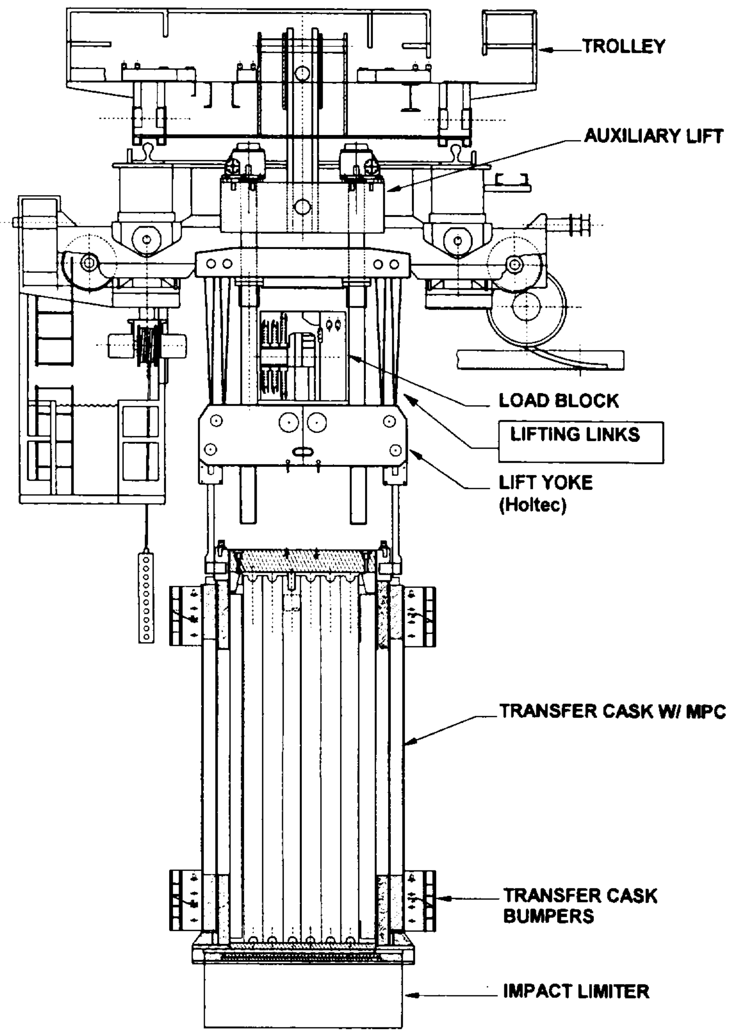
Notes: Main hook is engaged, but not shown
Final lid restrain will differ

MPC Lid Installation



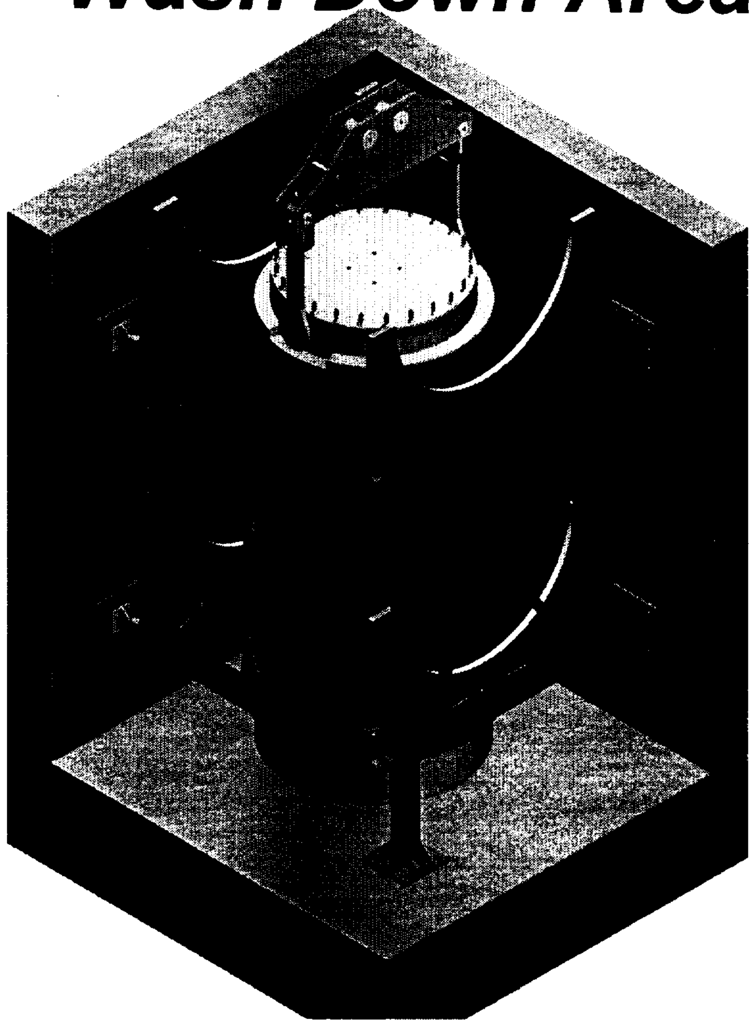
Note: Main hook is engaged, but not shown

TRANSFER CASK READY FOR HORIZONTAL MOVEMENT TO CWA



PG&E

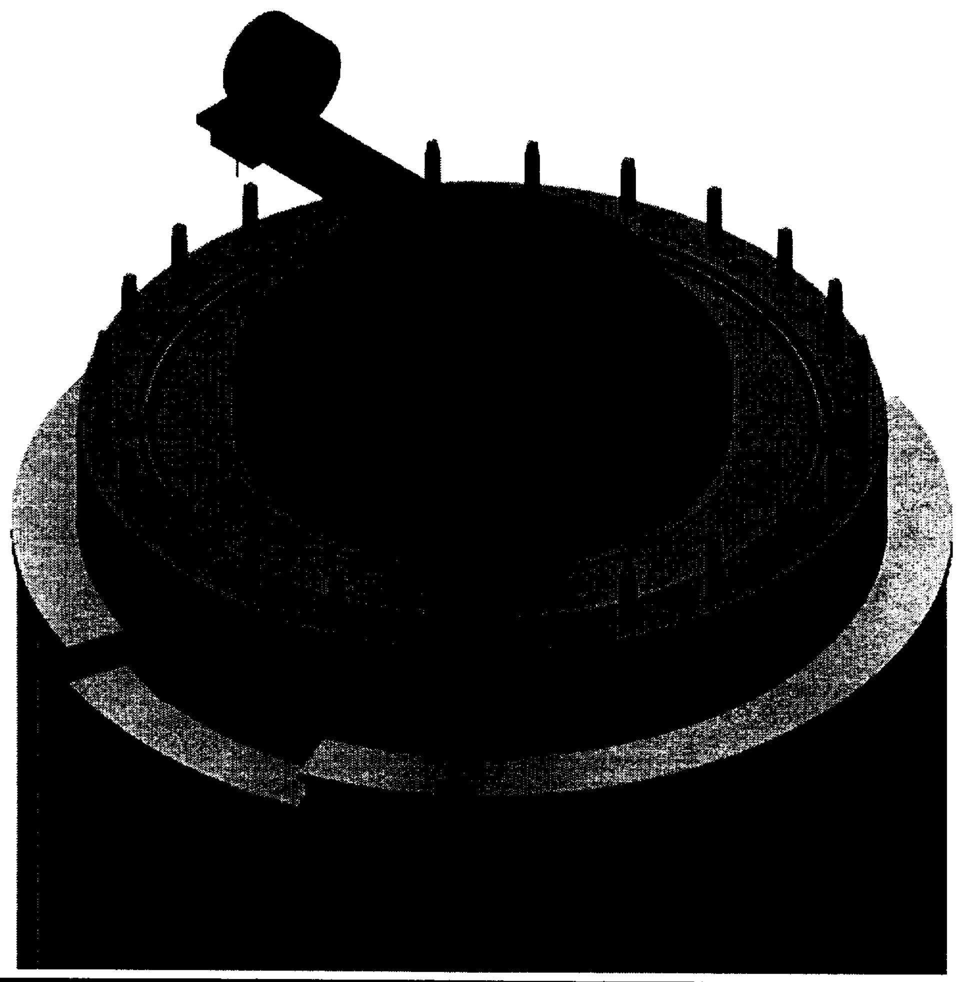
Placement of Transfer Cask in Cask Wash Down Area



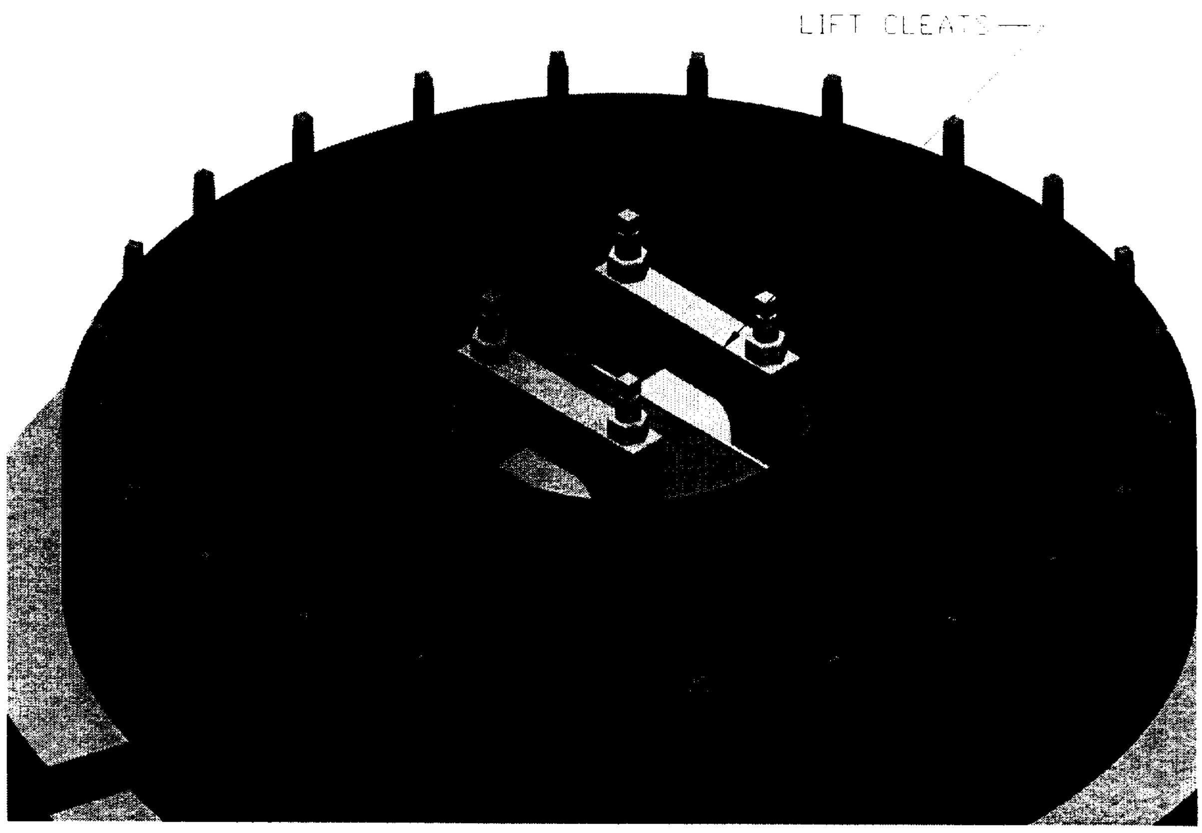
Note: Main hook is engaged, but not shown

Note: Seismic Restraint is not current version in this slide

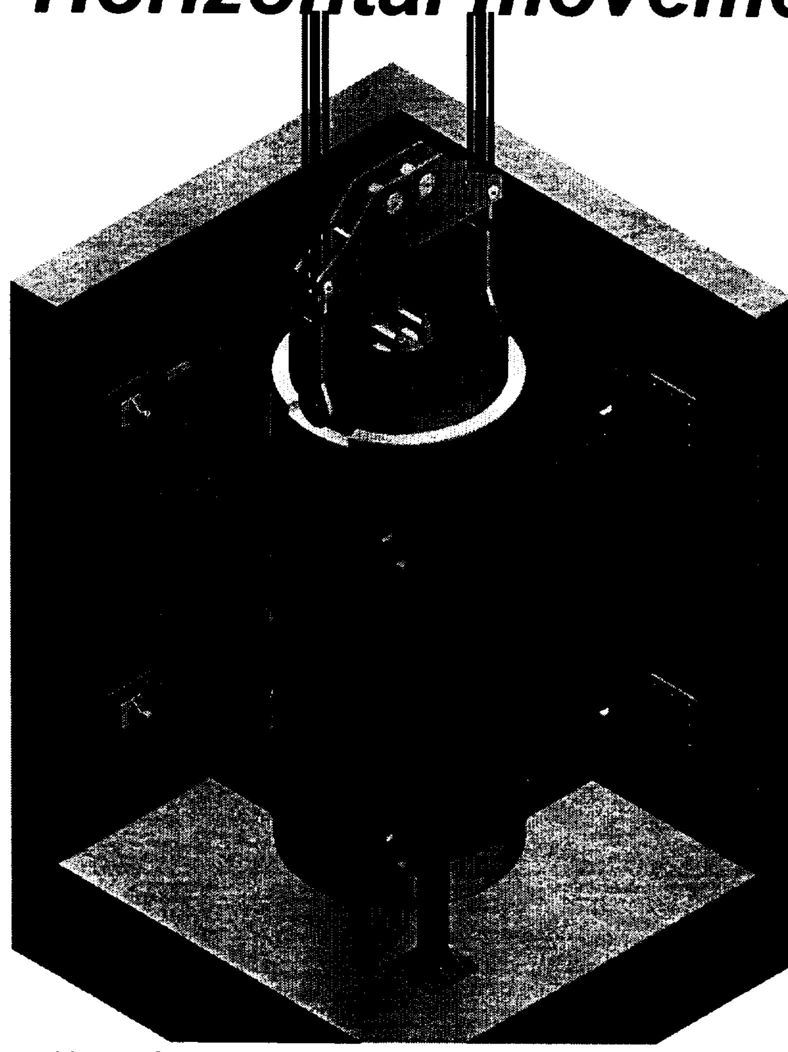
MPC Lid Welding



MPC Lift Cleat Installation



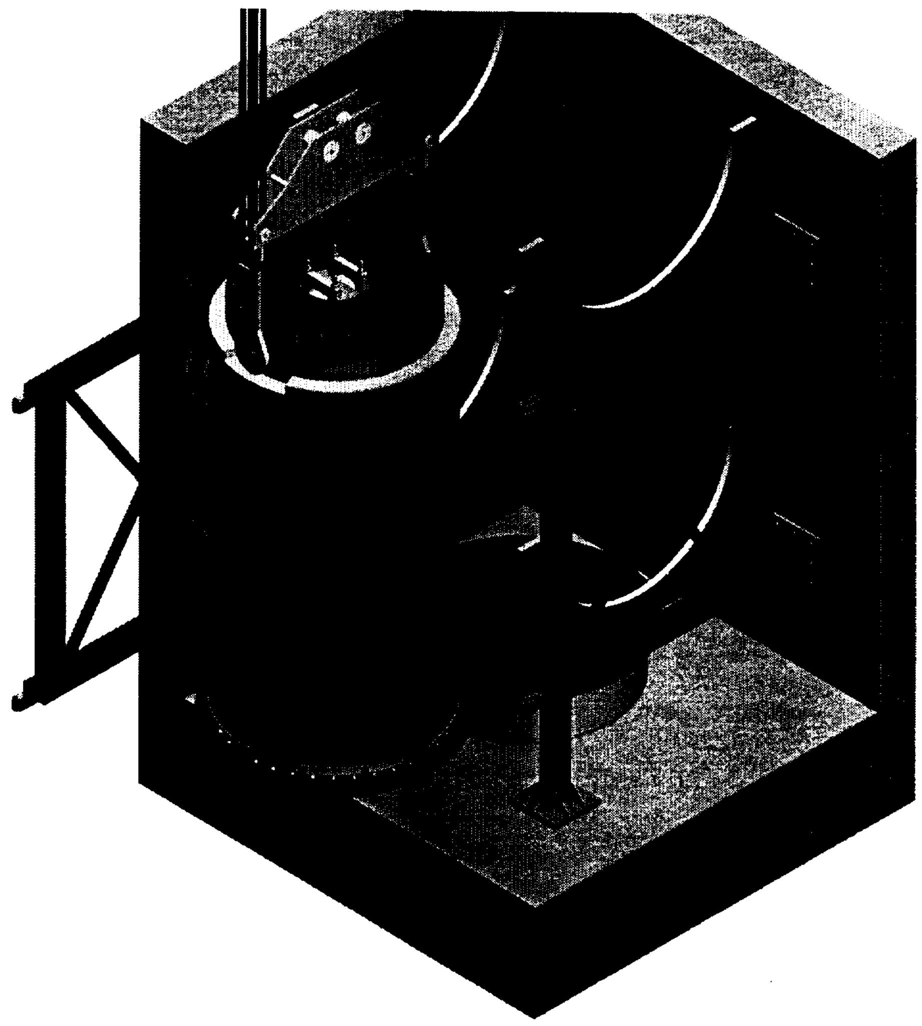
Transfer Cask Readied for Horizontal movement



Note: Main hook is engaged, but not shown

Note: Seismic Restrain is not current version in this slide

Transfer Cask Removal from Frame

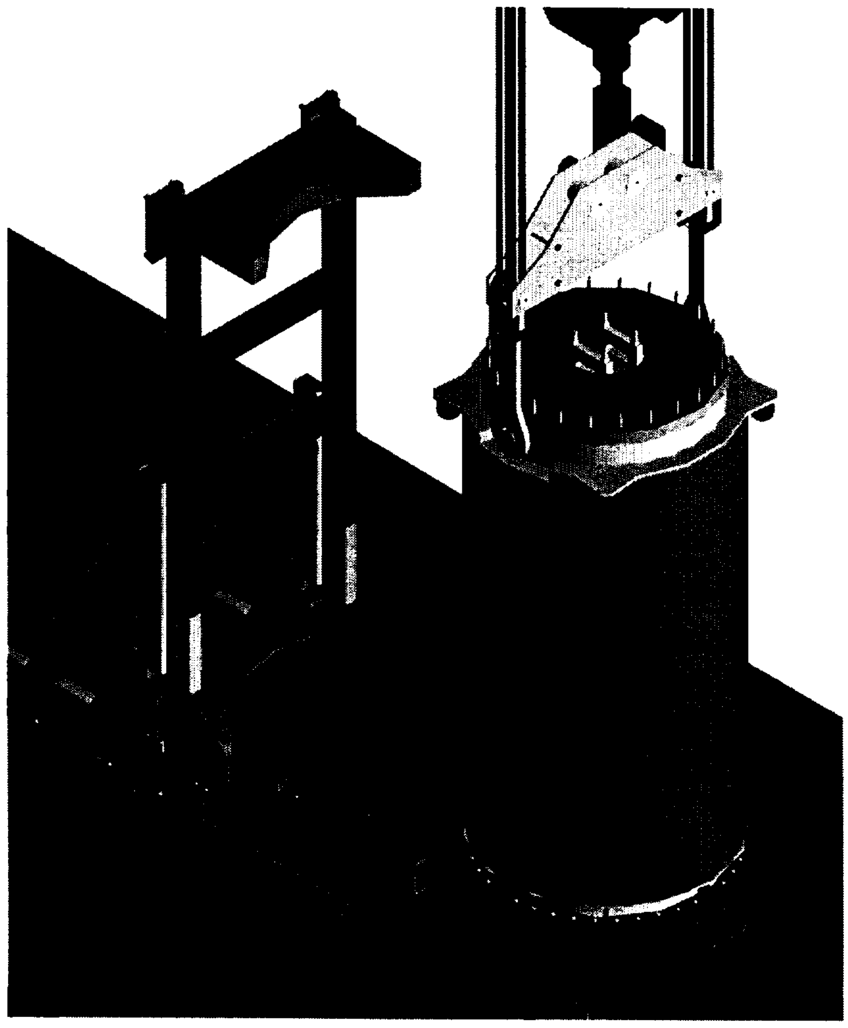


Note: Main hook is engaged, but not shown

Note: Seismic Restraint is not current version in this slide

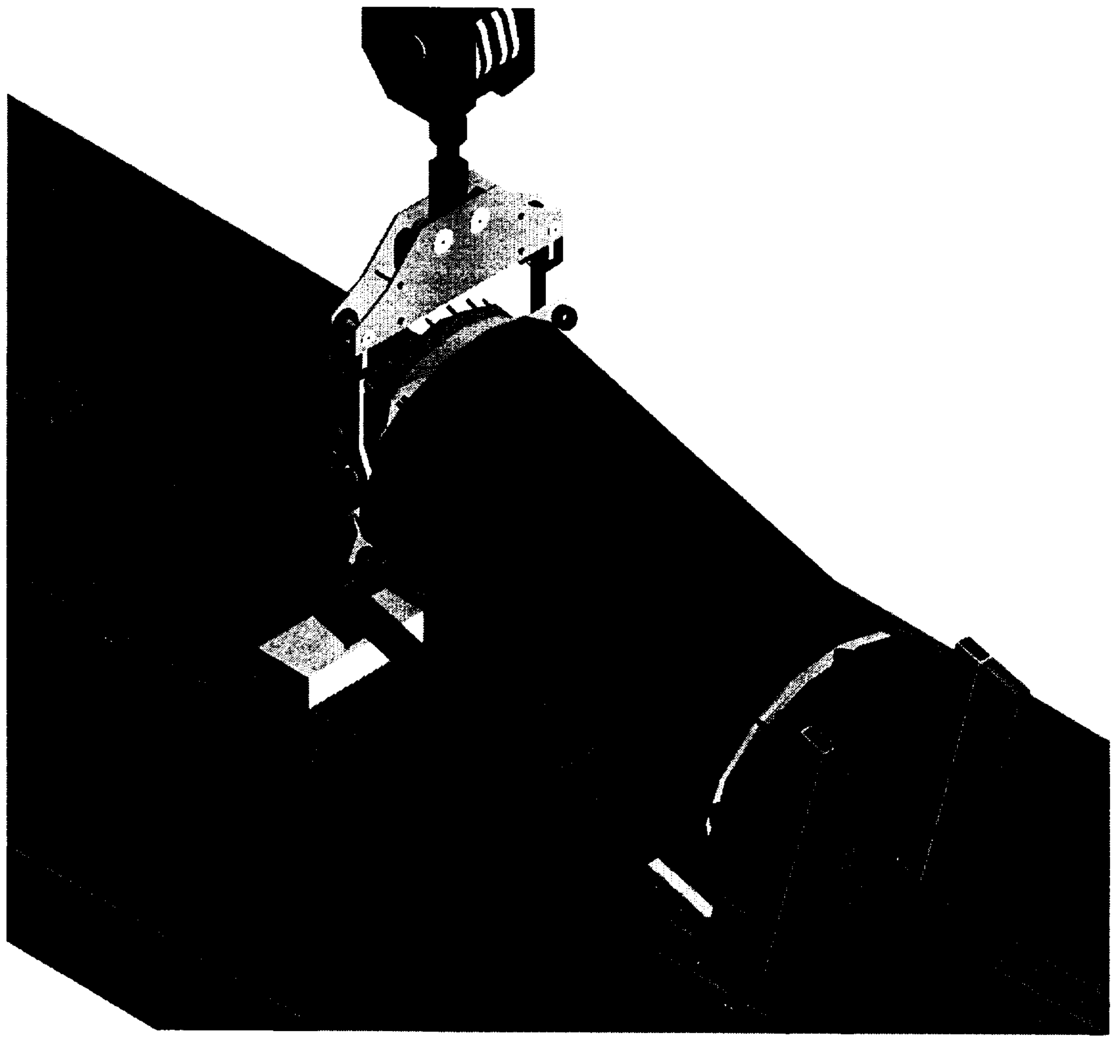
PG&E

Transfer Cask Placed on Transport Frame

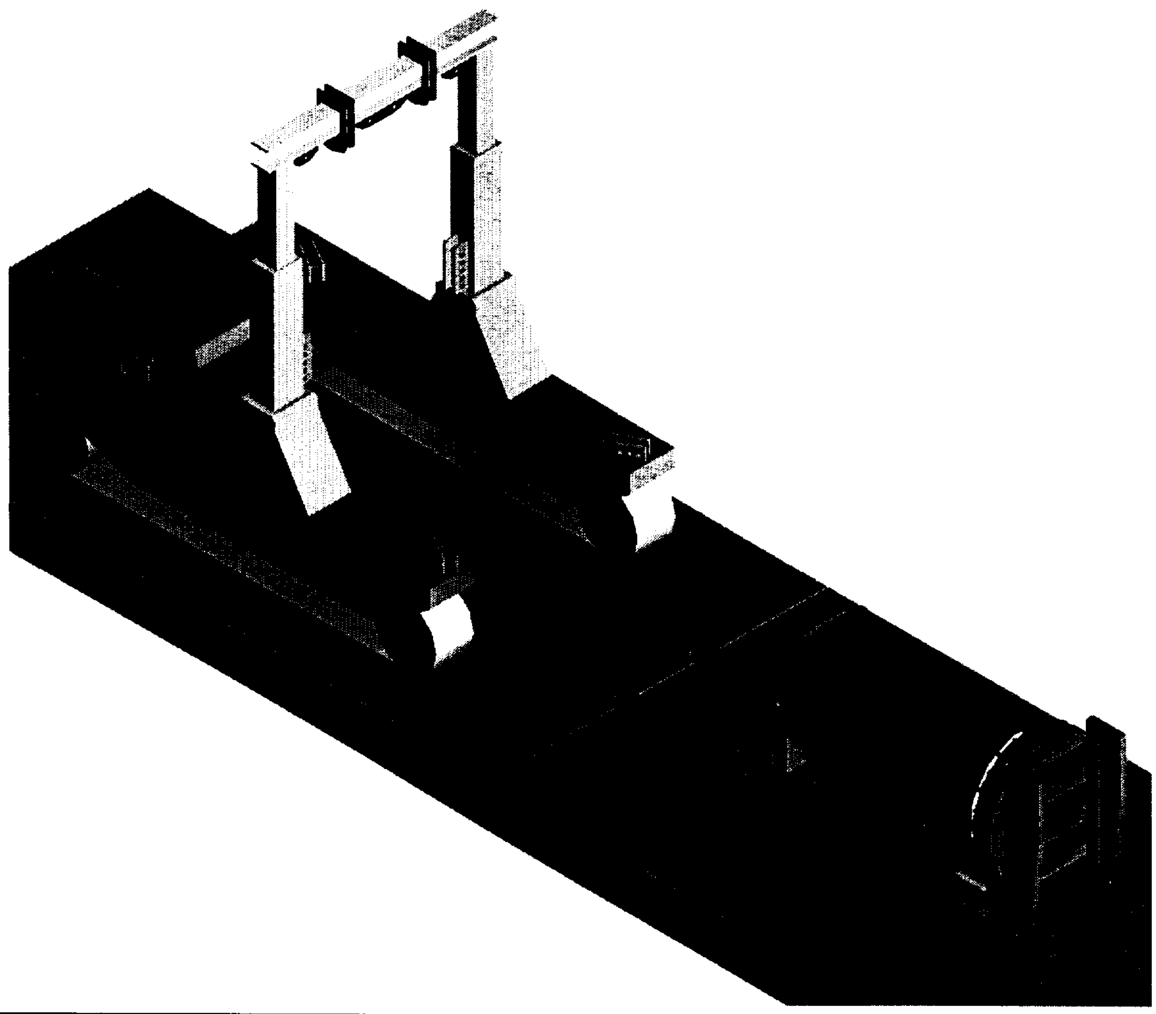


Note: Main hook is engaged, but not shown correctly

Transfer Cask Downending Using Impact Limiter

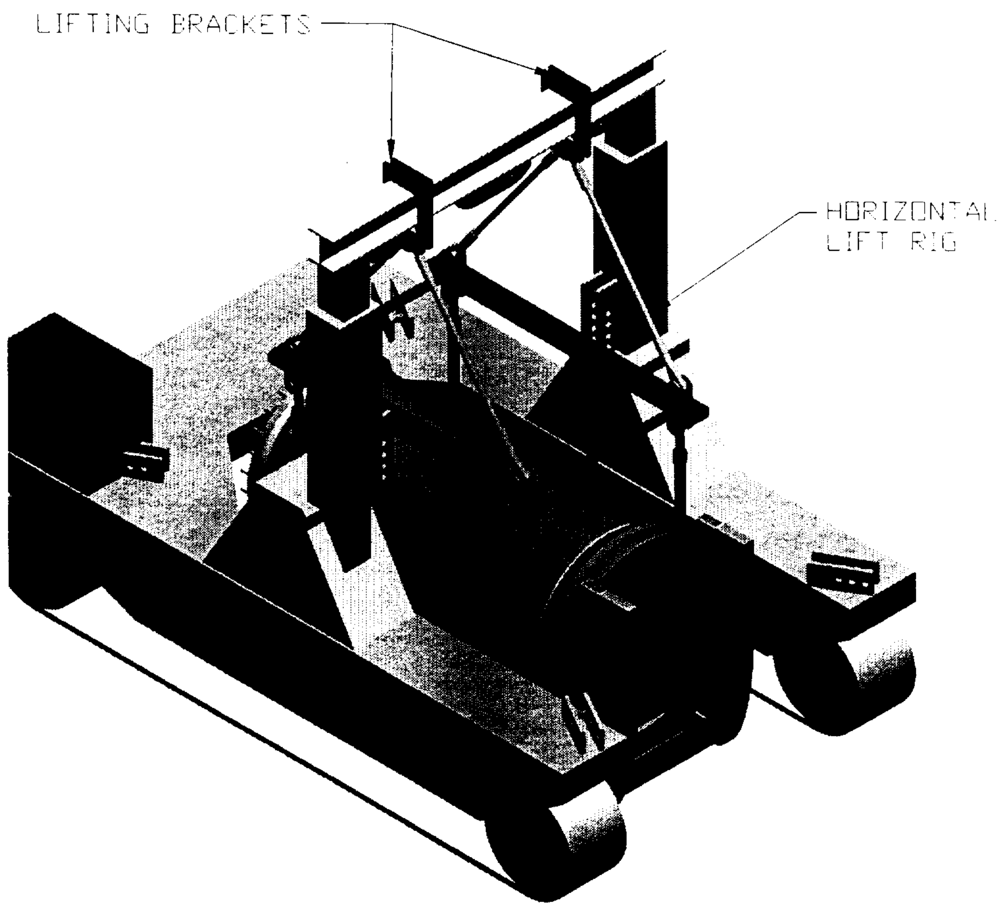


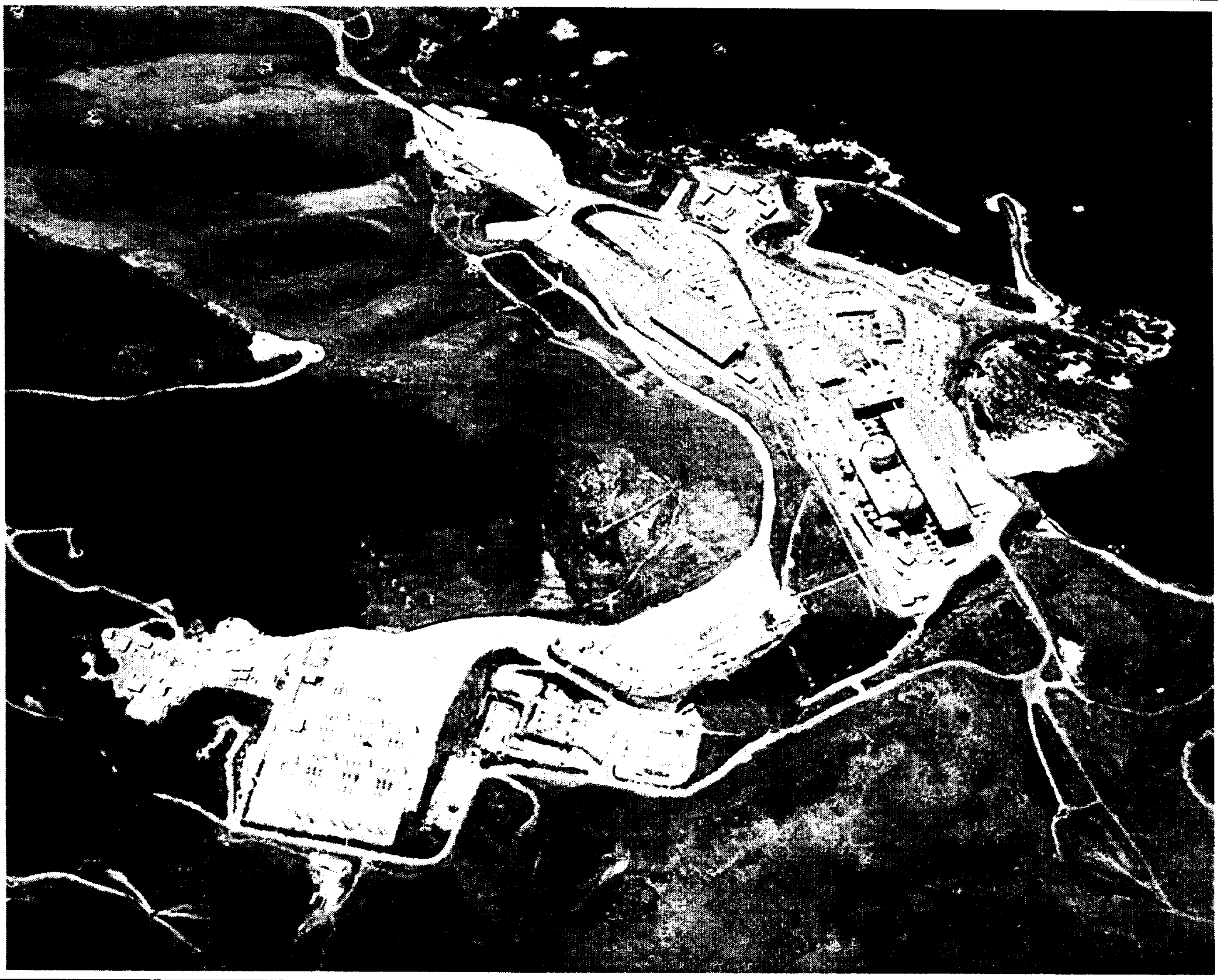
Transfer Cask movement to Transporter



PG&E

Transporter Rigging





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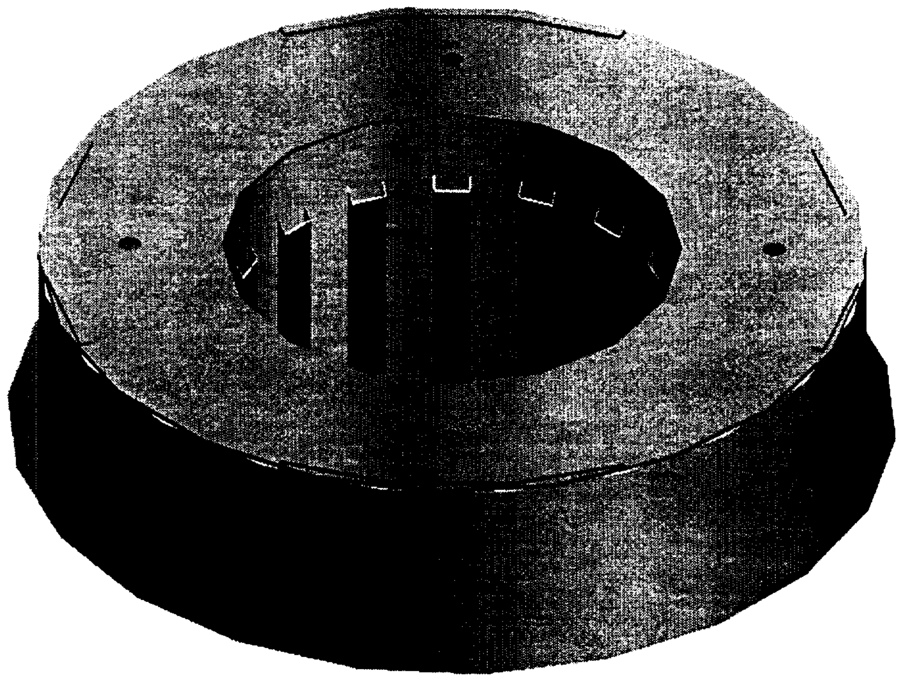
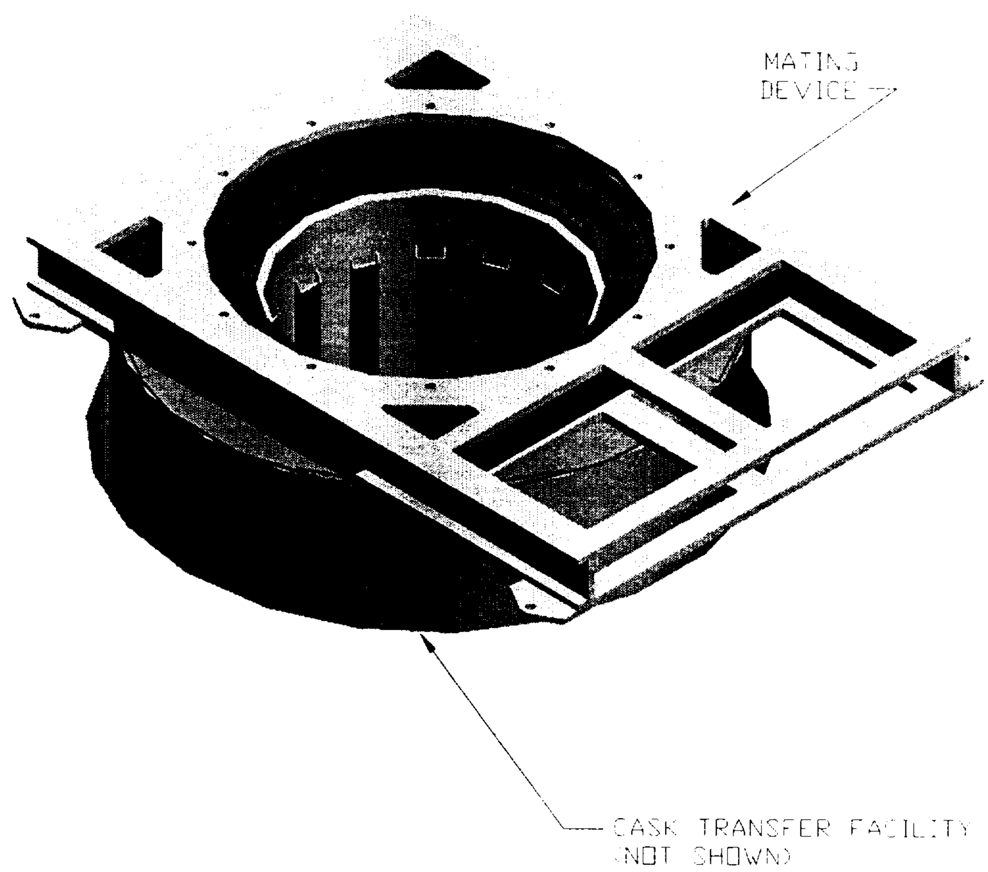
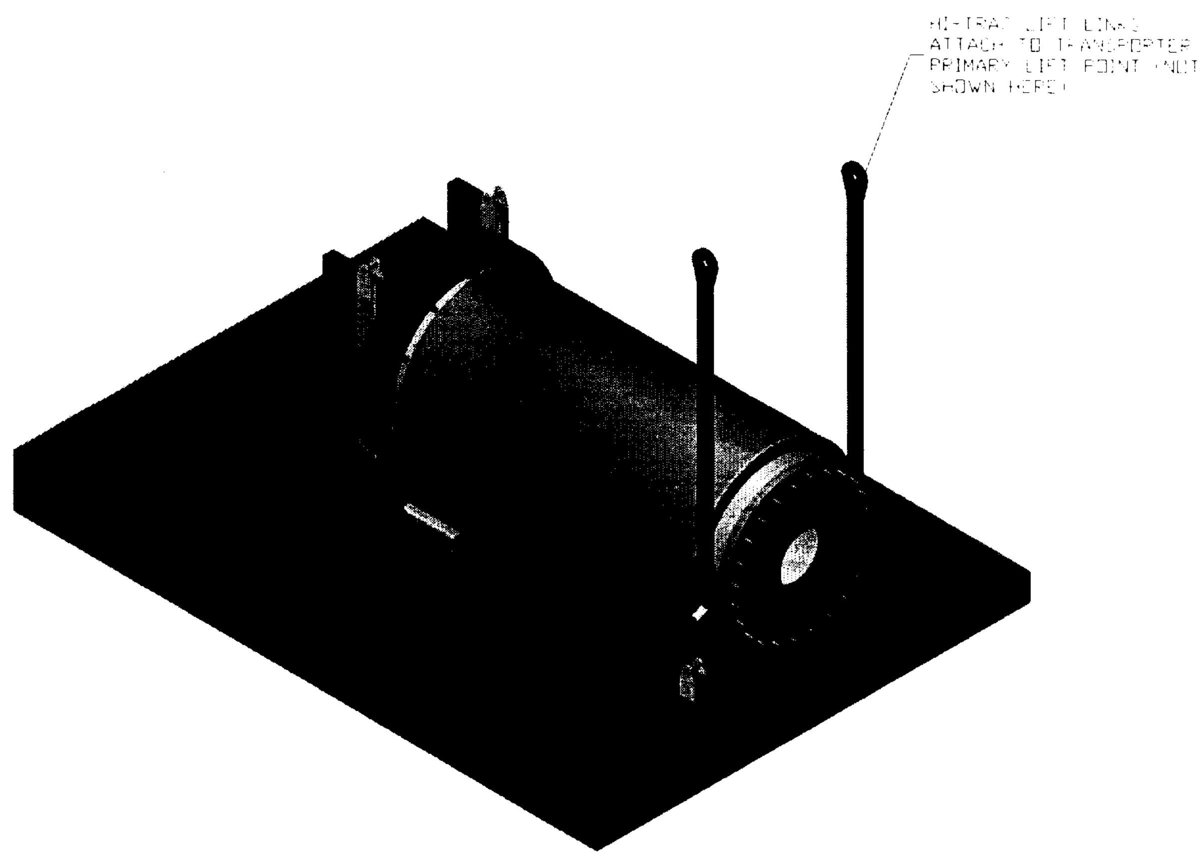
Hi-Storm Overpack Lowered into Cask Transfer Facility



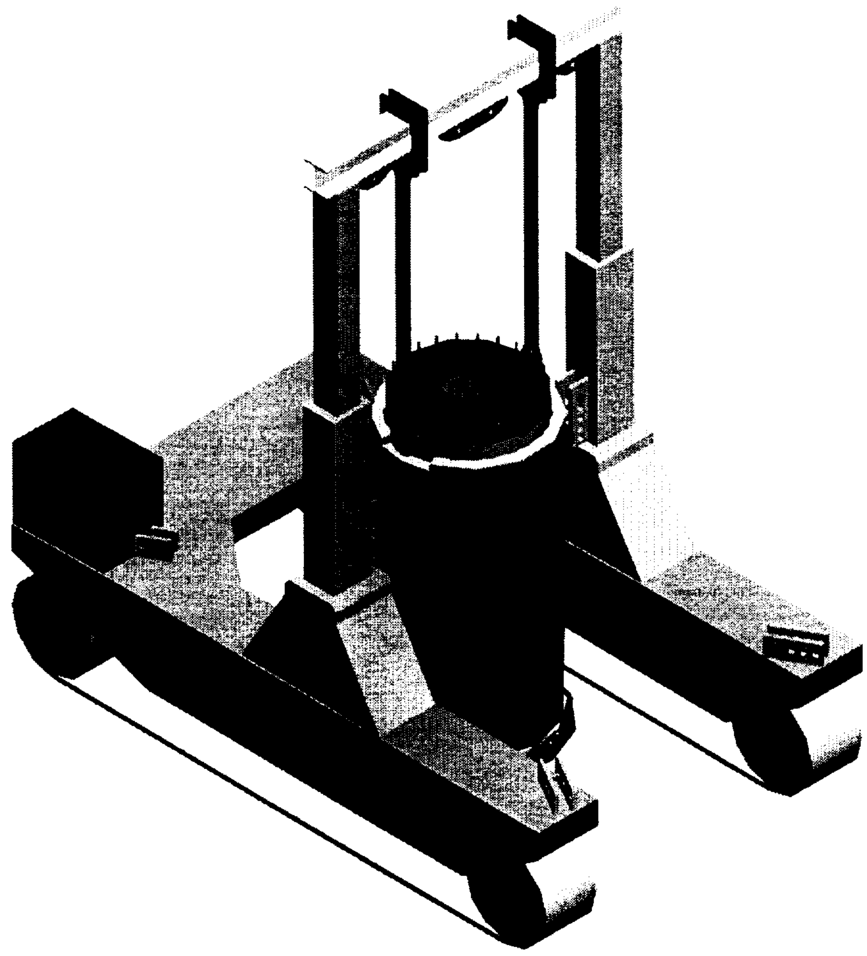
Hi-Storm Overpack with Mating Device



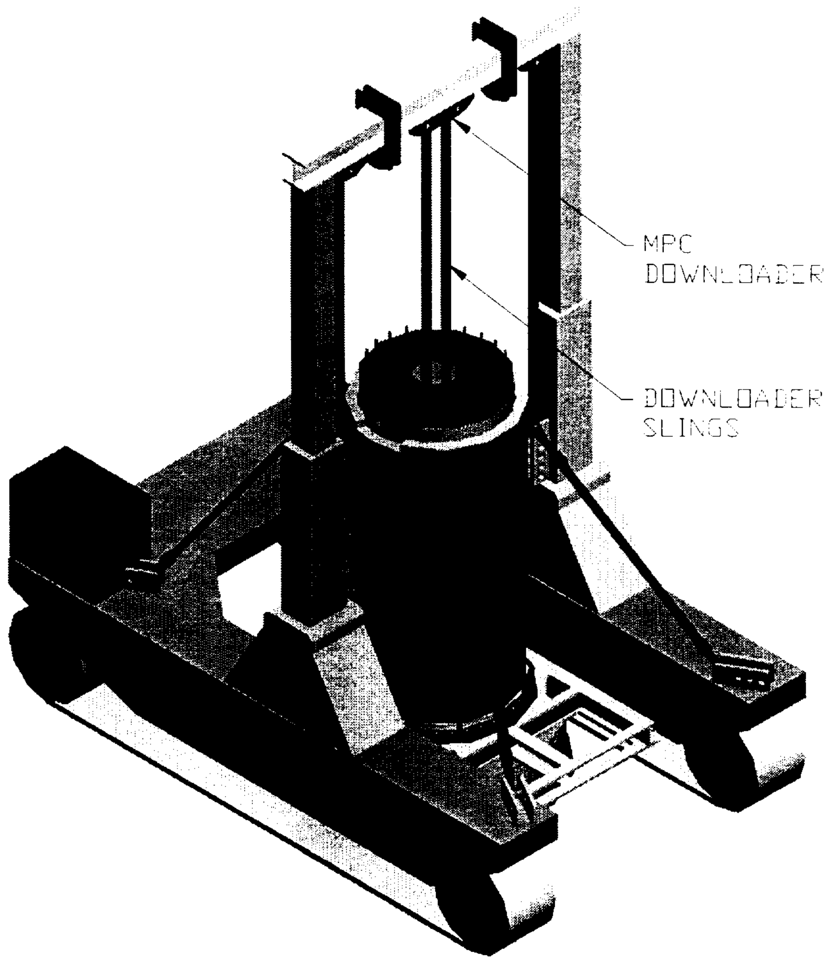
Transfer Cask Upended
(Transporter not Shown)



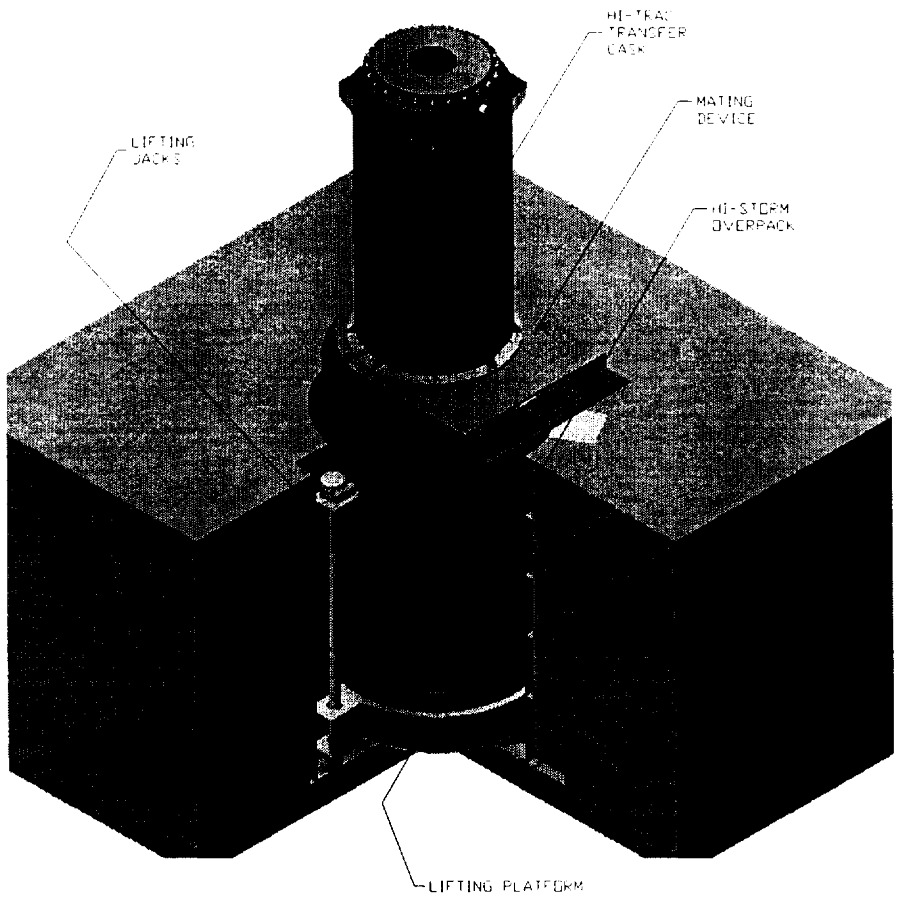
Transfer Cask movement to CTF



Placement of Transfer Cask over Storage Cask



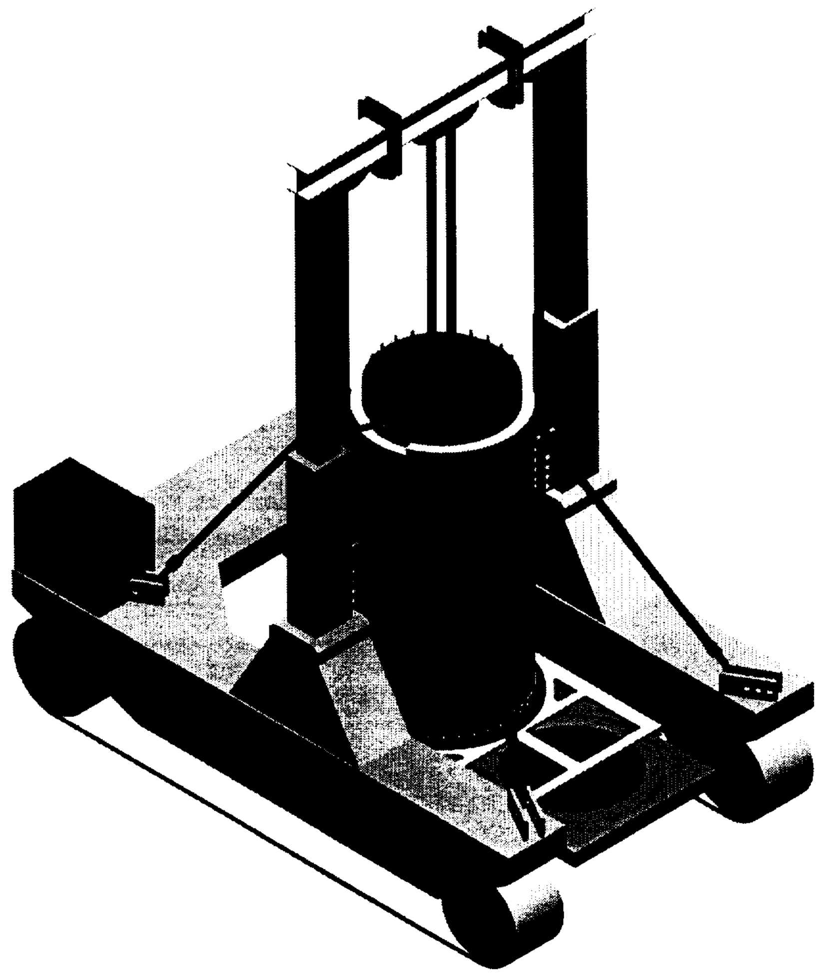
AT CTF W/O TRANSPORTER SHOWN



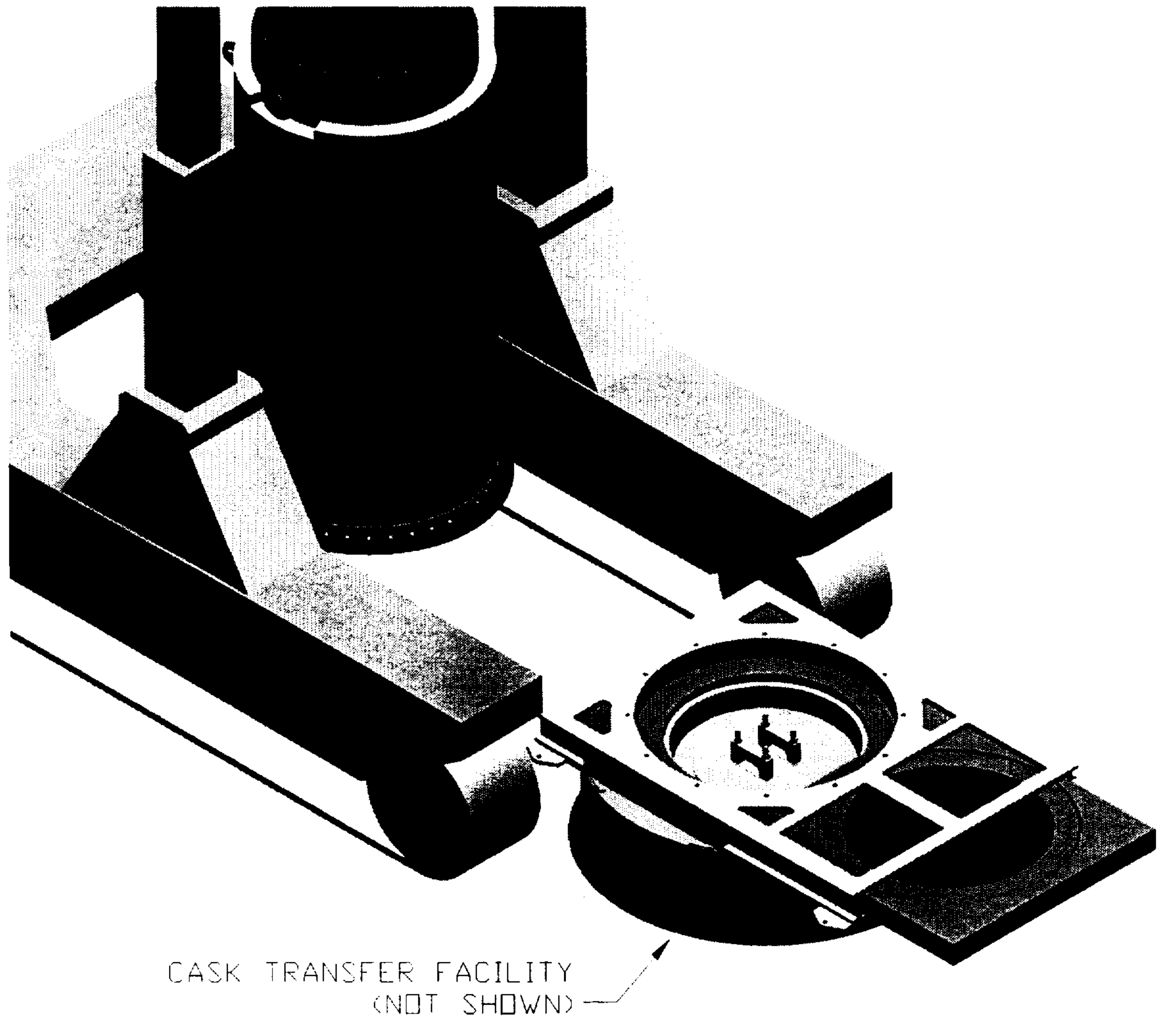
Note Transporter and lateral restraints not shown

PG&E

Transfer of MPC into Storage Cask

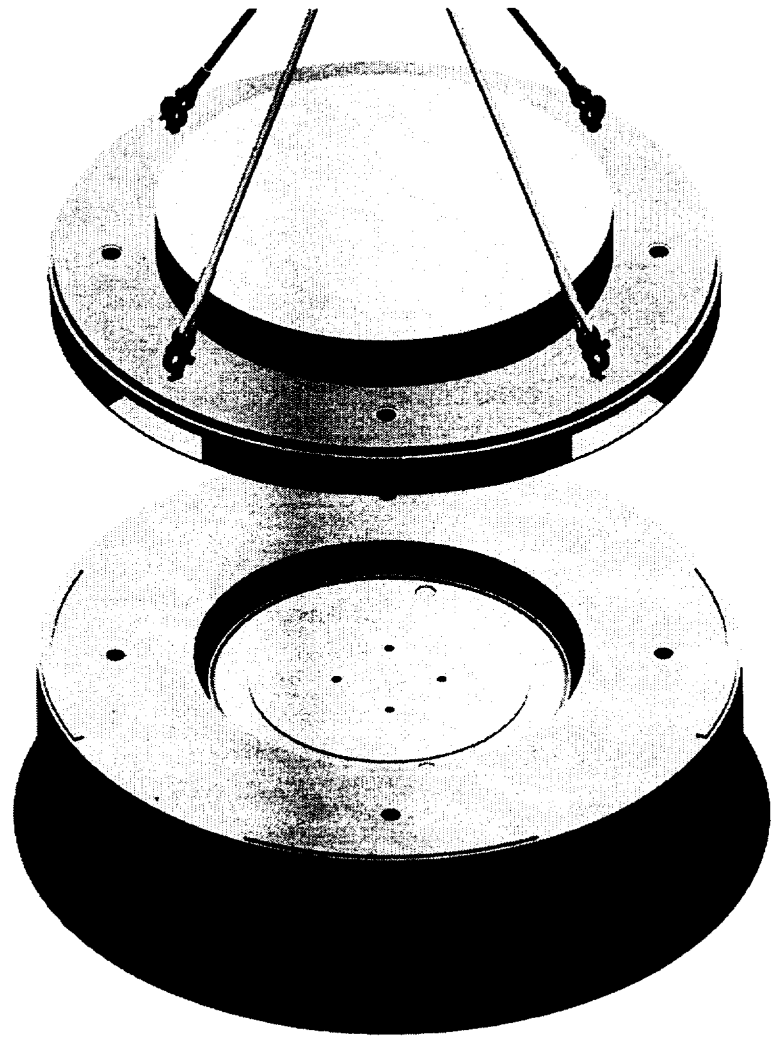


Transfer Cask Removal from CTF

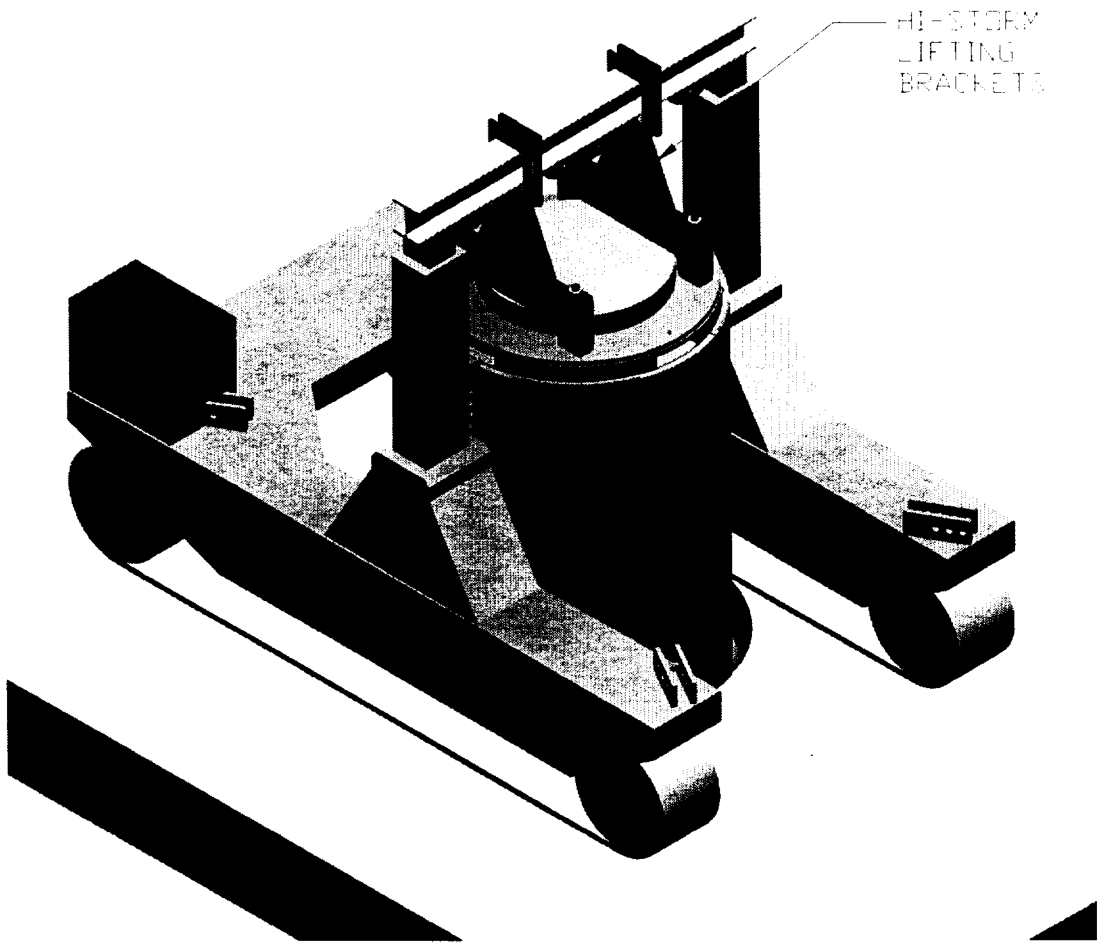


PG&E

Storage Cask Lid Placement

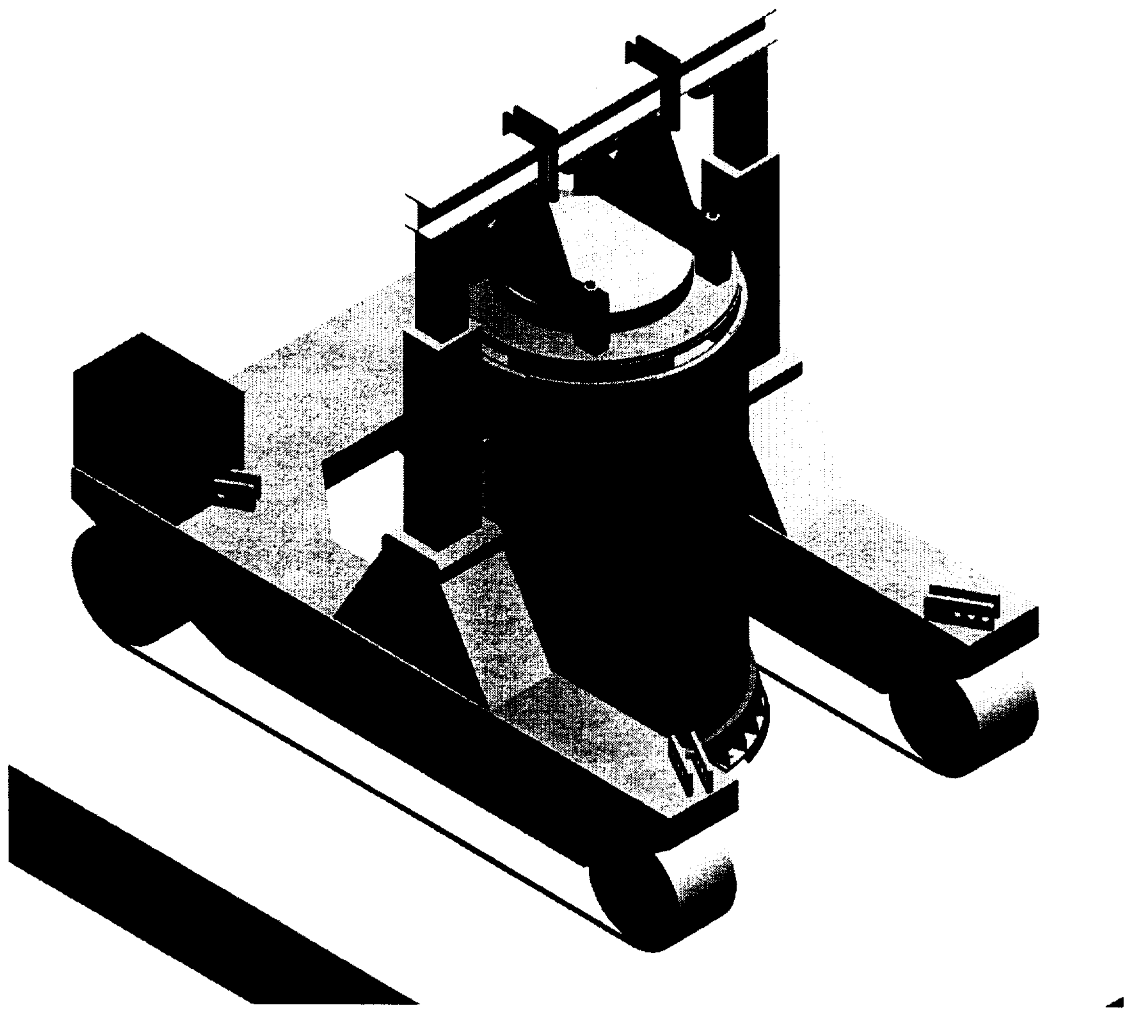


Overpack Raised out of CTF

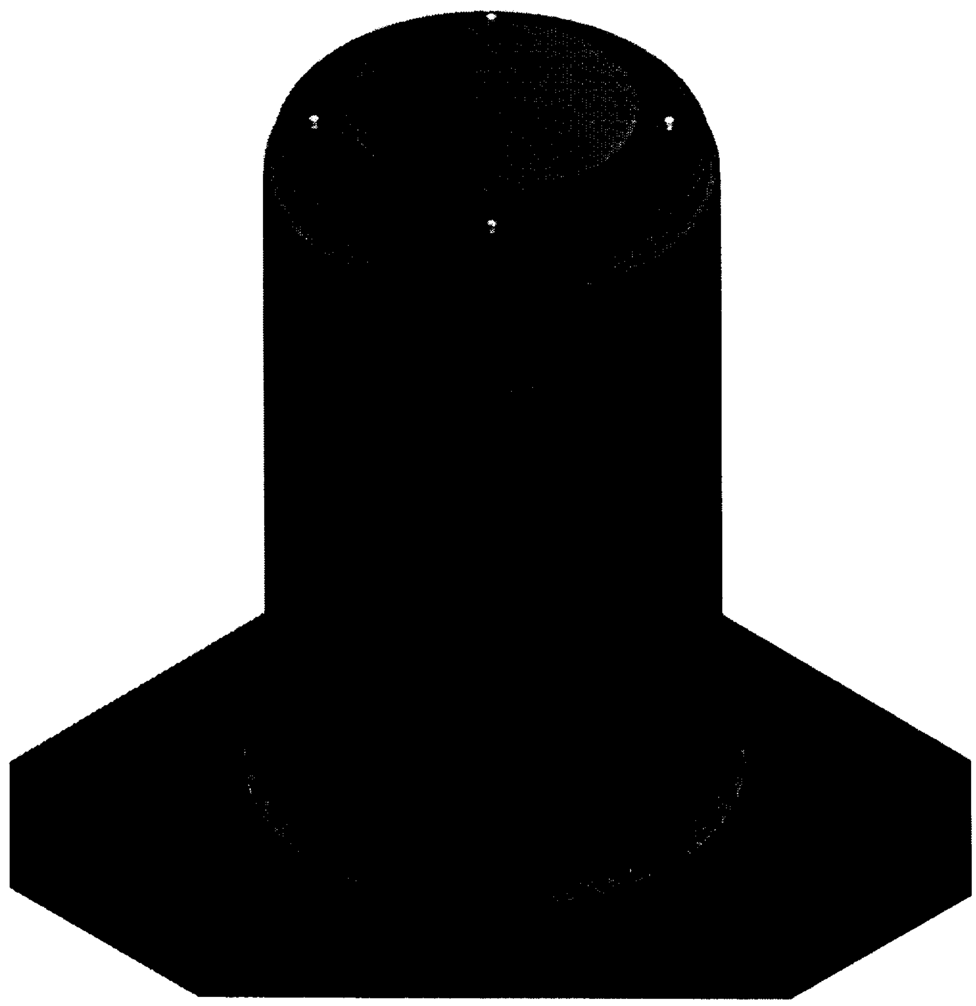


PG&E

Overpack Transported to Storage Facility



Overpack Placement on Storage Pad



Overpack Loading Operations

Activity	Part 50	Part 72
1. Move empty cask and MPC into FHB and prepare for loading	Impact on structure	N/A
2. Empty transfer cask and MPC being placed in SFP	Heavy load drop	N/A
3. Load fuel assemblies into MPC	Spent fuel movement in pool	Fuel TSs
4. Remove transfer cask from SFP	Heavy load drop on structure	Thermal req's
5. Decontamination	Existing processes	N/A
6. Welding, leak testing and prepare for movement	Releases and SSIP	Fuel conditions/ closure req's
7. Transfer cask movement in FHB	Heavy load drop on structure	N/A
8. Transfer cask movement outside FHB	Effect on plant SSCs	Transporter stability

Part 50 and 72 Scope

- Part 50
  - Crane modifications
  - Heavy load drop structural analyses
  - Cask seismic restraints
  - Affect on facility during transport
- Part 72(Holtec CoC 1014)
  - Cask structural limits (drops, missiles, etc)
  - Criticality analysis during cask handling
  - Thermal analysis during cask handling