CALCULATION COVER SHEET

FLSmidth

JOB NUMBER	CLIENT	PROJECT	CALC. NUMBER
11021	CC&V	High Grade Ore	C-002

TITLE/SUBJECT:

Reclaim Tunnel

STATEMENT OF PROBLEM:

Design Concrete Vault and Tunnel for Reclaim Area.

P.E. SEAL

COLORADO LICENSED
EDWARD CARTER SUMMERS
45828
7-23-12
PROFESSIONAL ENGINEER

SOURCES OF DATA, FORMULA & REFERENCES:

IBC 2009
ACI 318-08
Robot Structural Analysis Professional 2012
Handbook of Steel Drainage and Highway Construction Products (1994)

CONCLUSIONS & RECOMMENDATIONS:

See Calc for Results

REV	DATE	DESCRIPTION/STATUS	CALC BY	CHKD BY	DWG./SPECS REVISED	QA REVIEW
A	Sep-11	Ready for Check	ECS	SK		
0	Jul-12	Issued for Permit	ECS	SK		

FLSmidth

CALCULATION COVER SHEET

JOB NUMBER	CLIENT	PROJECT	CALC. NUMBER
11021	CC&V	High Grade Ore	C-002

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STATEMENT OF PROBLEM:

Design Concrete Vault and Tunnel for Reclaim Area.

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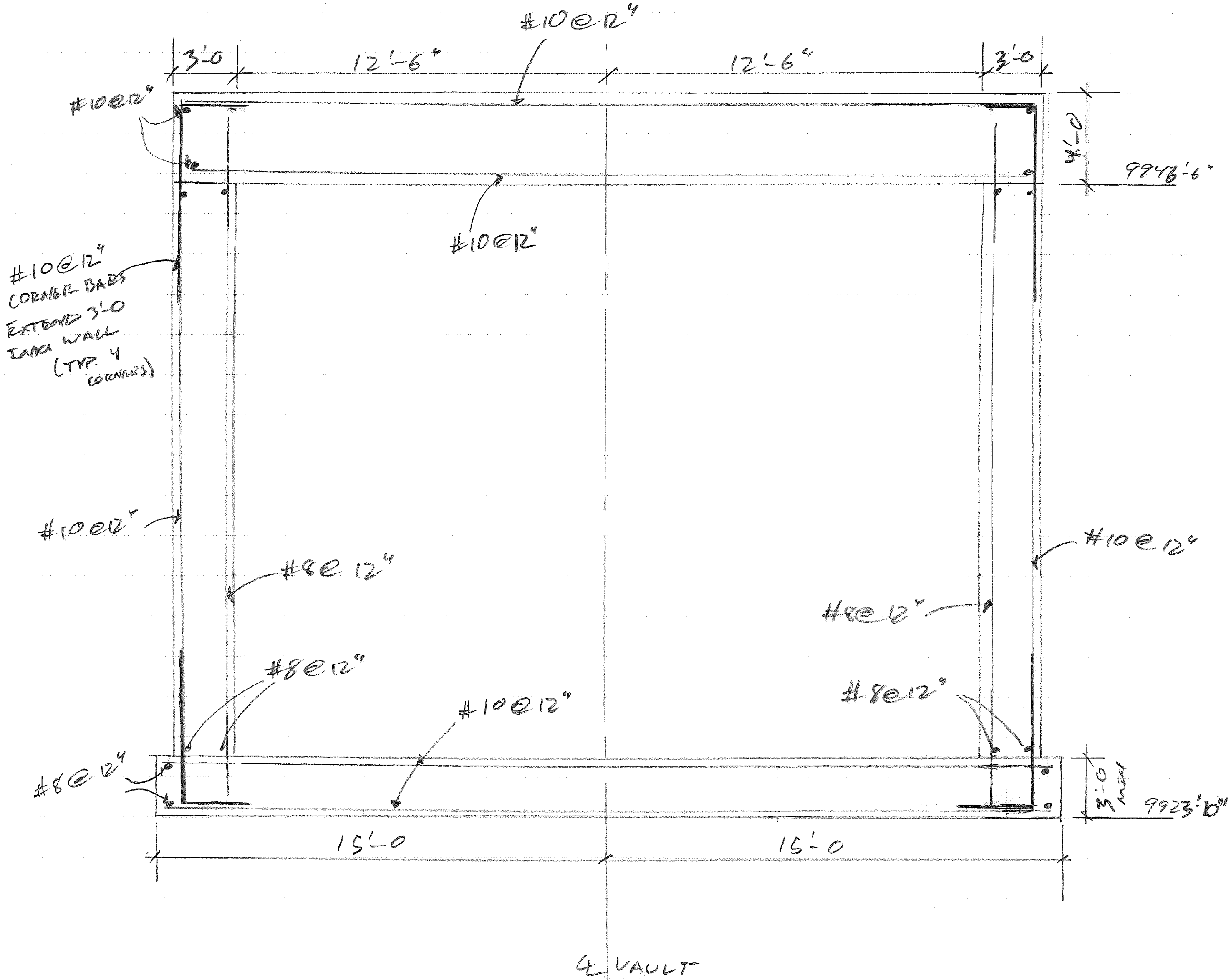
Handbook of Steel Drainage and Highway Construction Products (1994)

CONCLUSIONS & RECOMMENDATIONS:

See Calc for Results

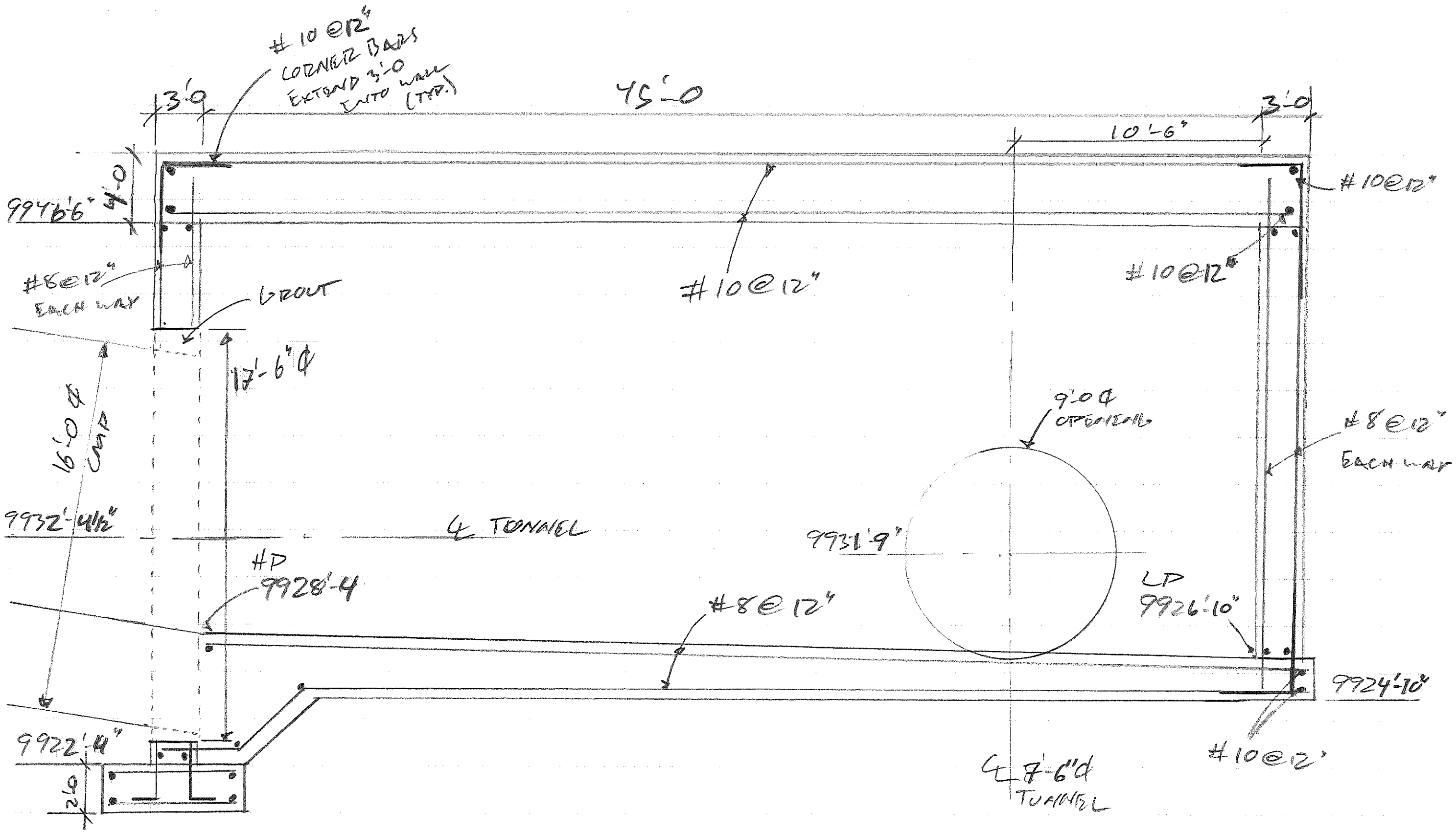
REV	DATE	DESCRIPTION/STATUS	CALC BY	CHKD BY	DWG./SPECS REVISED	QA REVIEW
A	Sep-11	Ready for Check	ECS	SK		
0	Jul-12	Issued for Permit	ECS	SK		

FLSmidth	PROJECT CC&V High Grade Ore	PROJ NO. 11021
		CALC NO. C-002
	SUBJECT Reclaim Vault Section	BY ECS DATE 9-16-11
		CHK DATE
		SHEET OF REV B

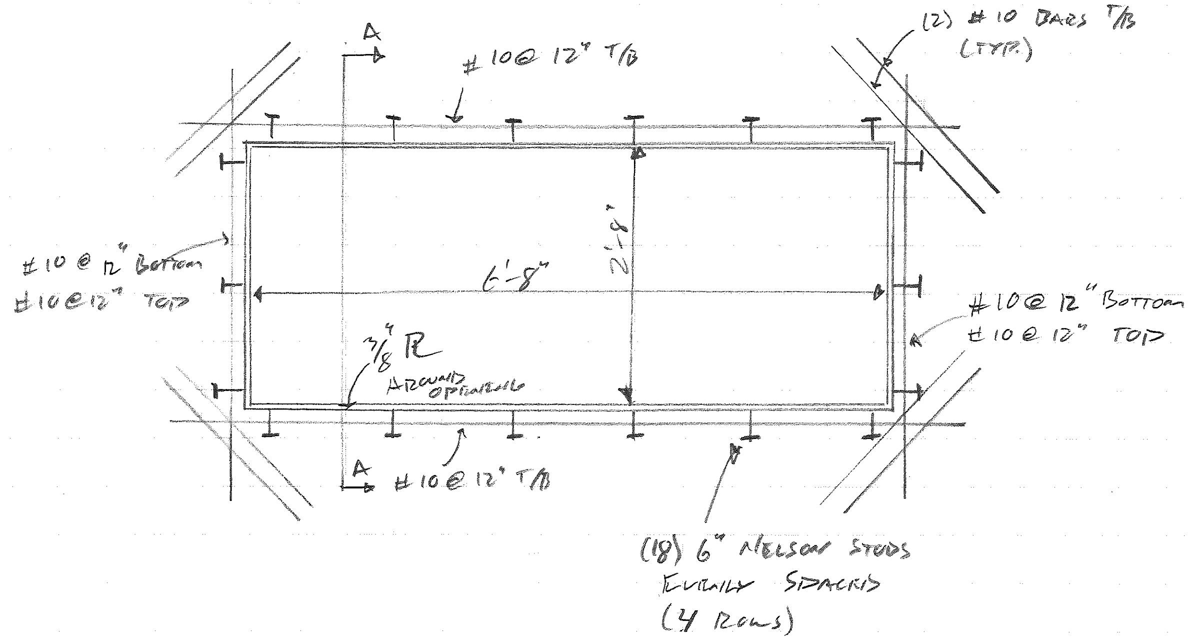


Reclaim Vault Section

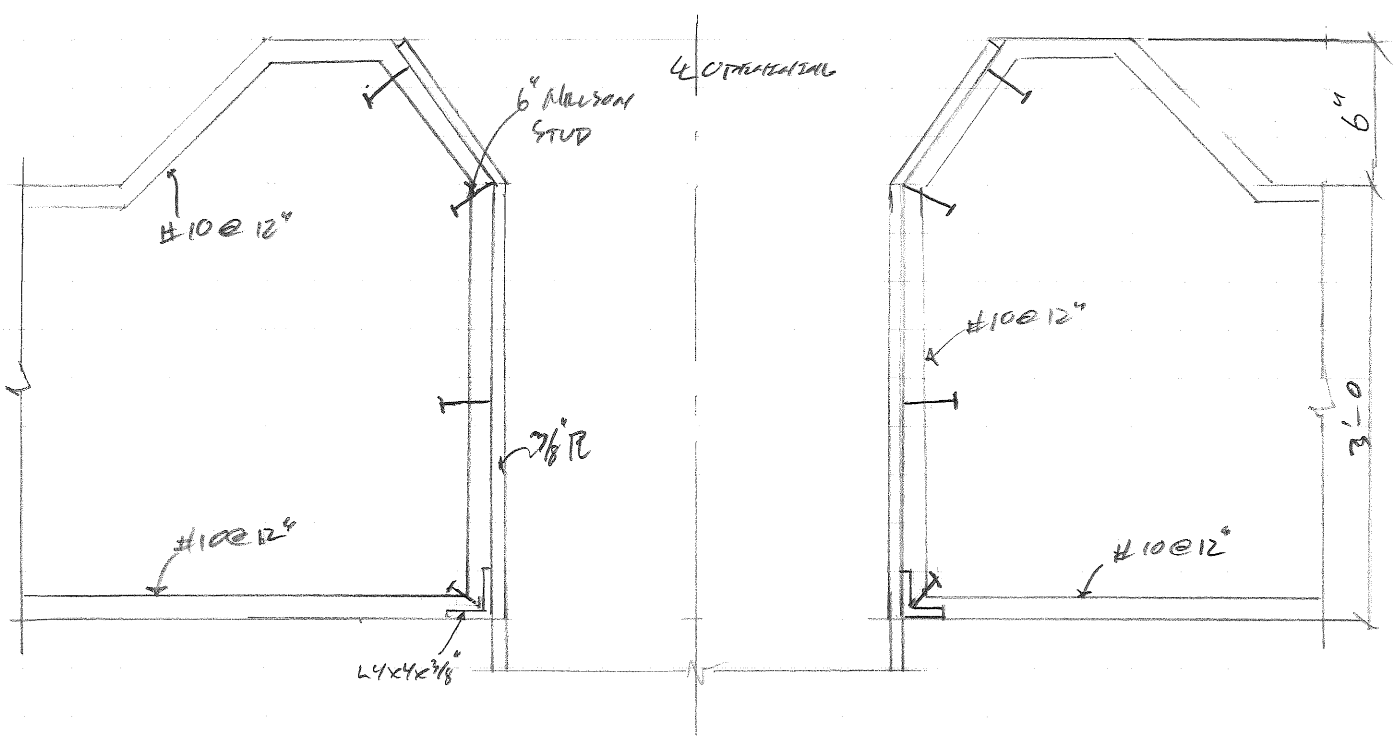
FLSMIDTH	PROJECT CC & V High Grade Ore	PROJ NO. 11021
		CALC NO. C-002
	SUBJECT Reclaim Vault Typical Section	BY ECS DATE 9-16-11
		CHK DATE
		SHEET OF REV B



FLSMIDTH	PROJECT CC & V HIGH GRADE ORE	PROJ NO. 11021	
		CALC NO. C-002	
	SUBJECT RECLAIM VAULT CHUTE OPENING	BY ECS	DATE 9-19-11
		CHK	DATE
		SHEET OF	REV B

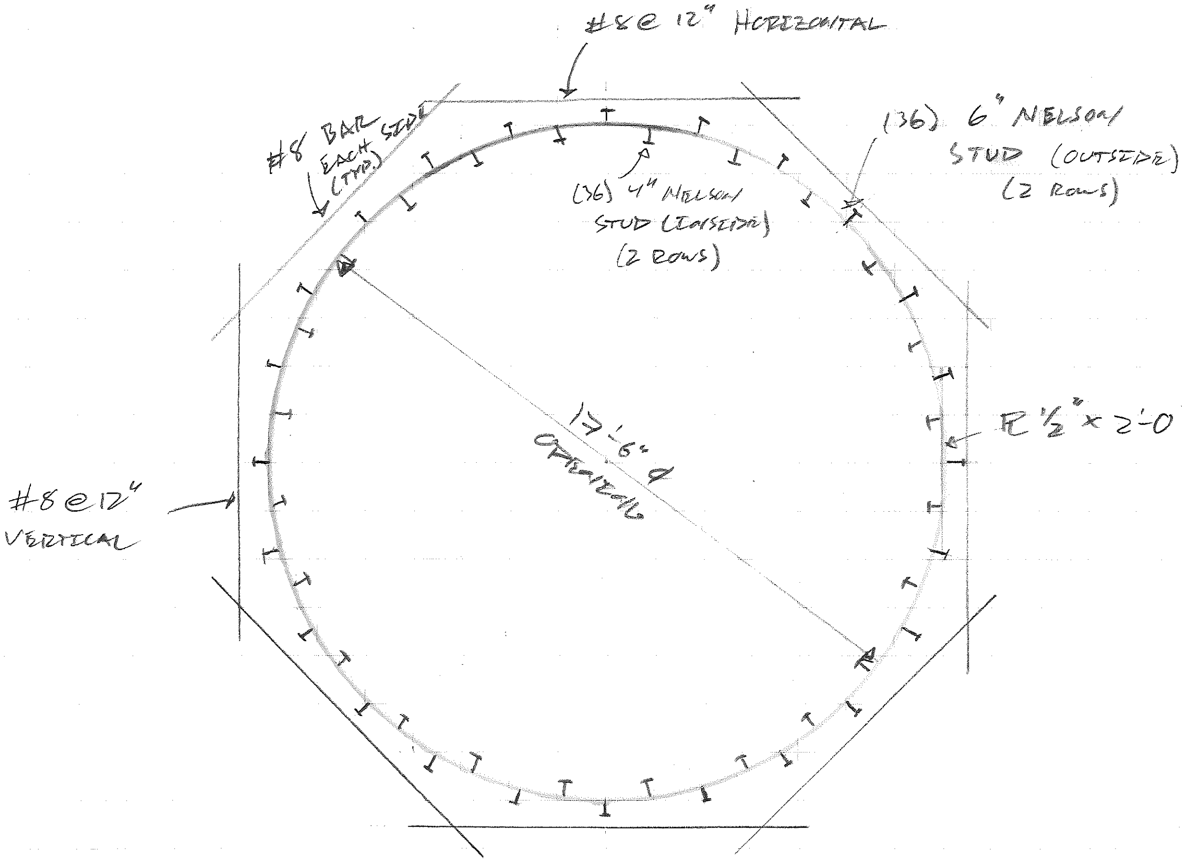


PLAN - CHUTE OPENING



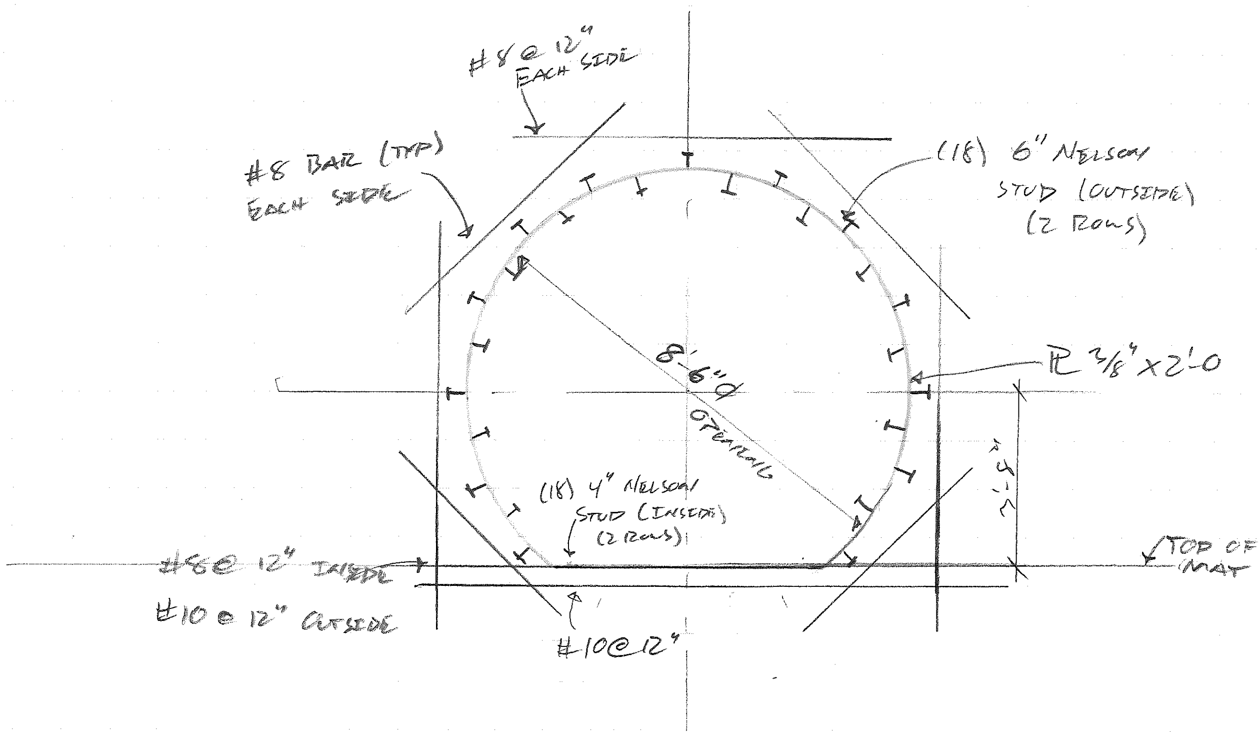
A-A

FLSMIDTH	PROJECT CC&V High Grade Ore	PROJ NO. 11021
		CALC NO. C-002
	SUBJECT Reclaim Vault	BY ECS DATE 9-19-11
		CHK DATE
		SHEET OF REV B



TUNNEL OPENING (16'-0 ϕ)

FLSmidth	PROJECT CC&V High Grade Ore	PROJ NO. 11021
		CALC NO. C-002
	SUBJECT Reclaim Vault	BY ECS DATE 9-19-11
		CHK DATE
		SHEET OF REV B



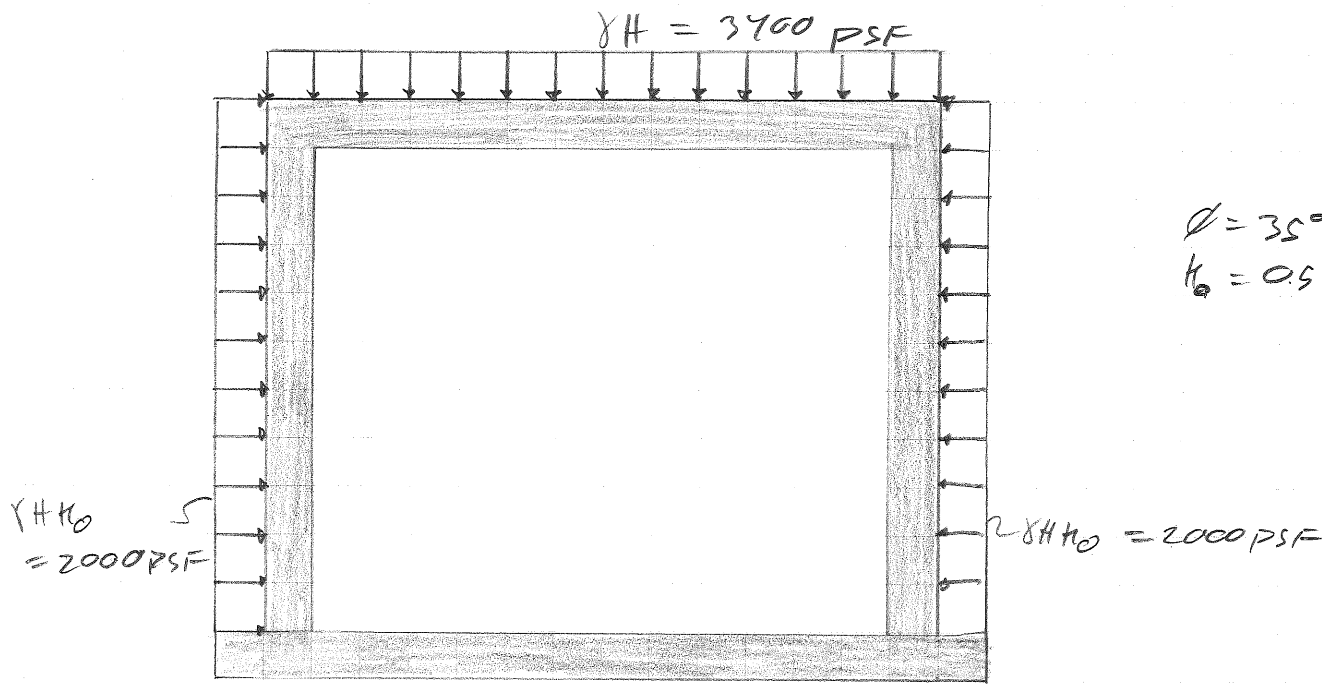
TUNNEL OPENING (7'-6"Ø)

Loads:

1. Reclaim Pile: (Surcharge Load)

γ = 85 pcf

H = 40'-0



ϕ = 35°

K_0 = 0.57

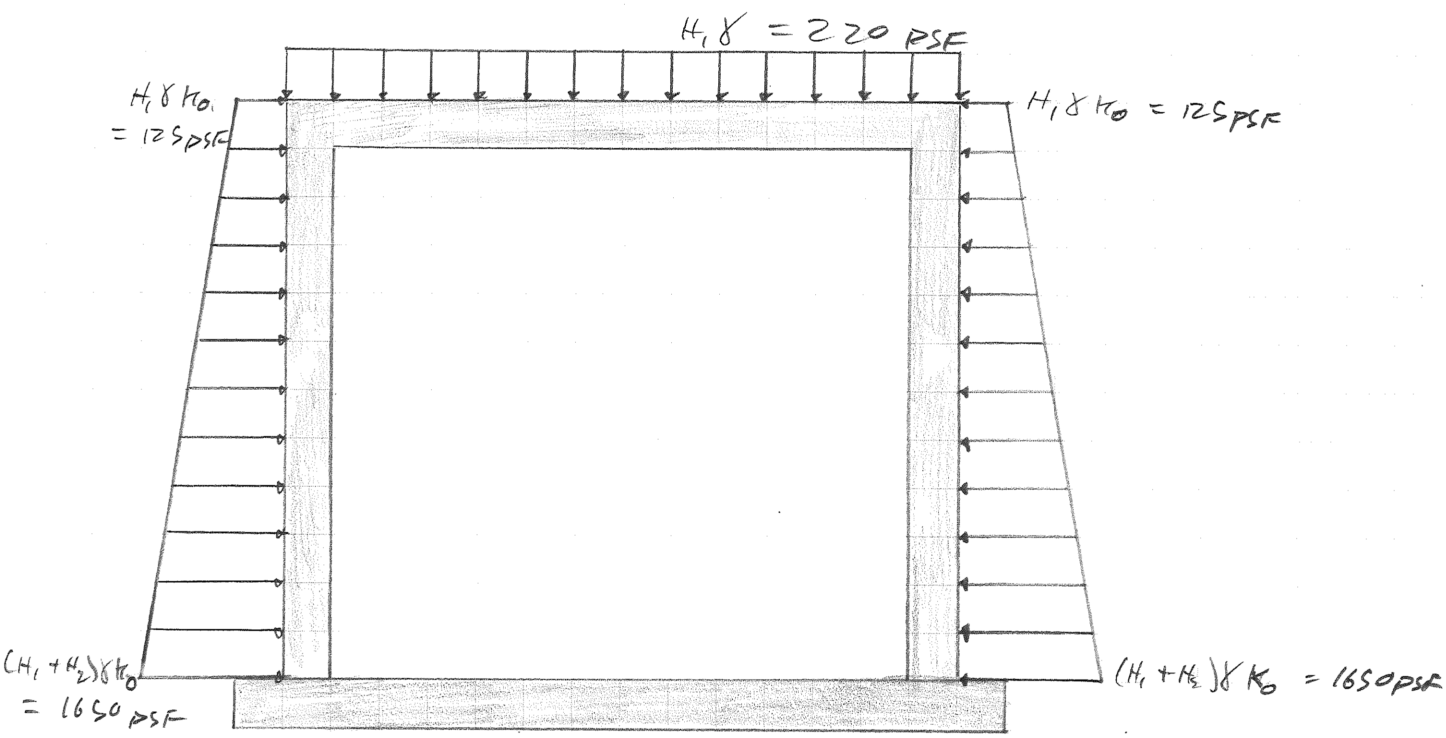
LOADS

2. SOIL BACKFILL

$\gamma = 110$ PCF (ASSUMED)
$\phi = 35°$

$K_0 = 1 - \sin\phi = 0.574$

$H_1 = 2'-0$ (HEIGHT ABOVE CONCRETE)
$H_2 = 24'-0$ (HEIGHT TO FOOTING)



	PROJECT CC&V HIGH GRADE ORE	PROJ NO. 11021
FLSMIDTH		CALC NO.
	SUBJECT RECLAIM TUNNEL VAULT	BY ECS DATE 9-9-11
		CHK DATE
		SHEET OF REV

LOADS:

3. SPEED BAR:

ASSUME WT 6x29

$(6)(5'-0)(29\#) = 870\#$

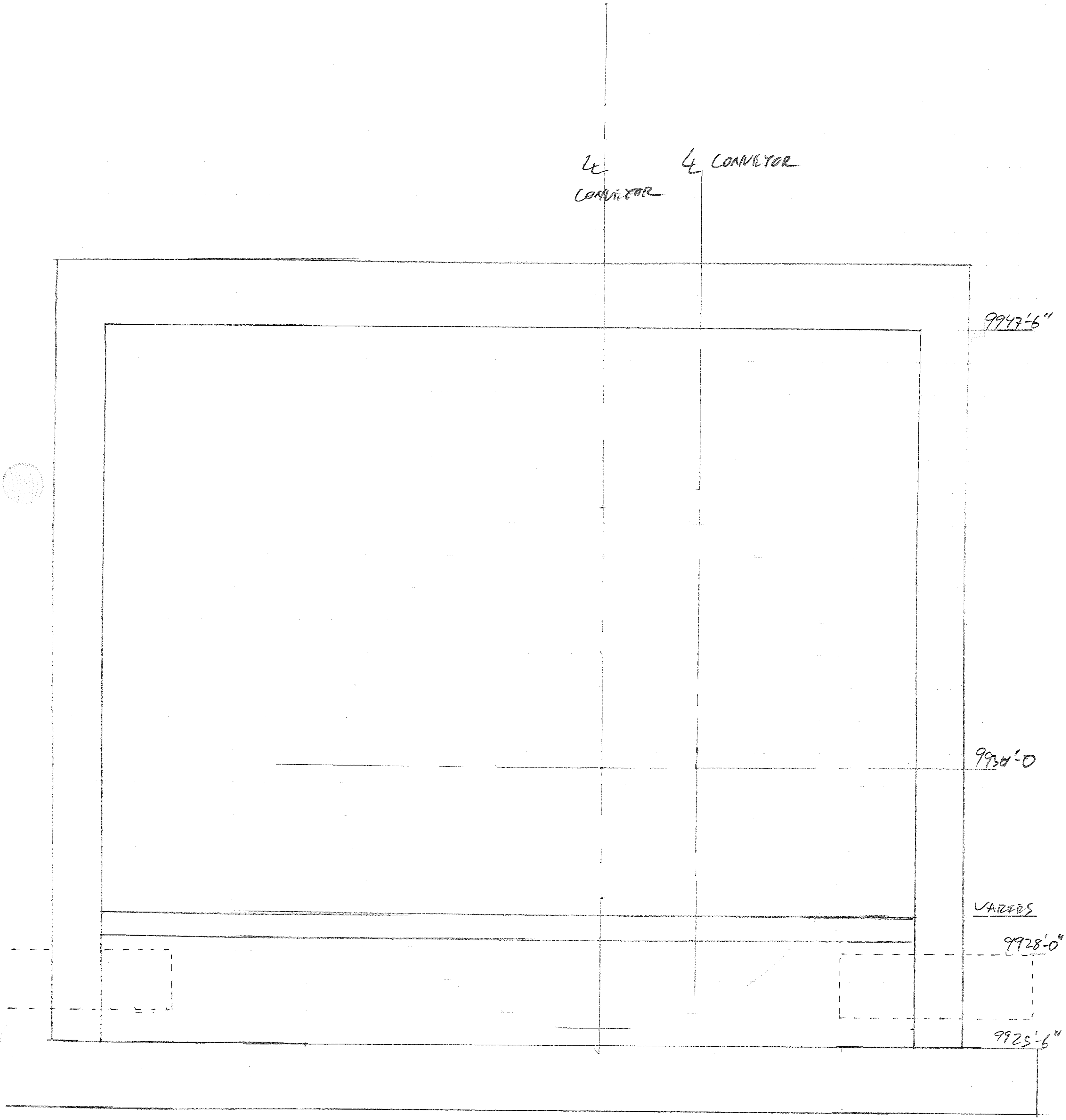
ASSUME 60 PLF FOR SPEED BAR FRAME

$(2)(4'-2" + 2'-3")(60 PLF) = 780\#$

$DL = 870\# + 780\# = 1650\#$ USE $\underline{2000}\#$

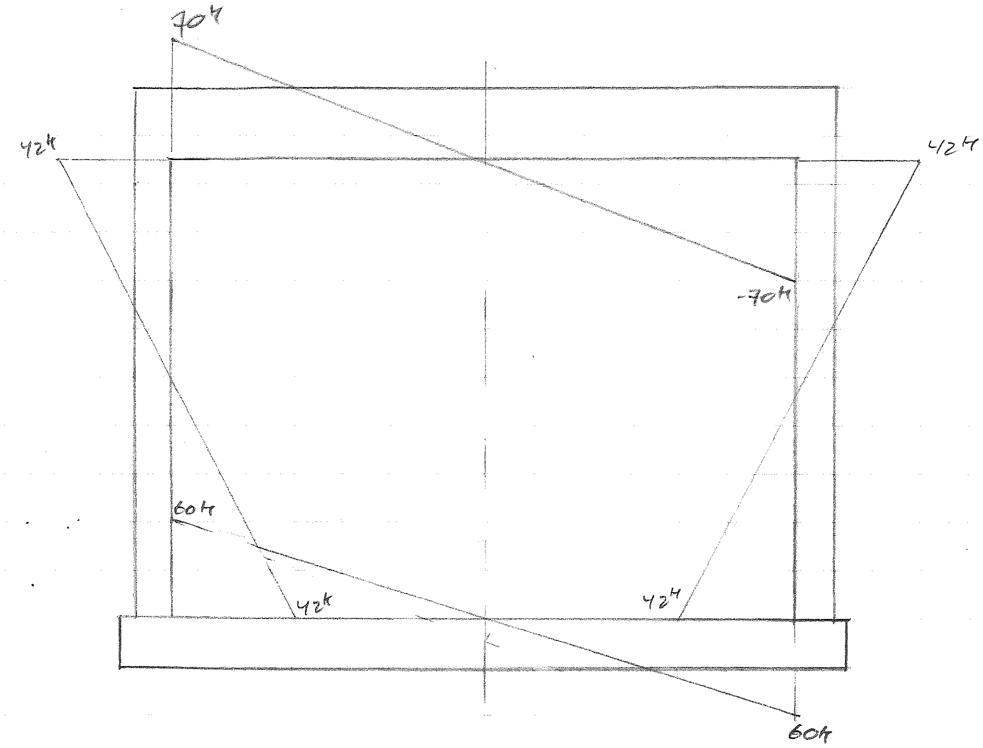
OR $\underline{150 PLF}$

FLSMIDTH	PROJECT CC&V HIGH GRADE ORE	PROJ NO. 11021
		CALC NO.
	SUBJECT RECLAIM TUNNEL VAULT	BY ECS DATE 9-9-11
		CHK DATE
		SHEET OF REV

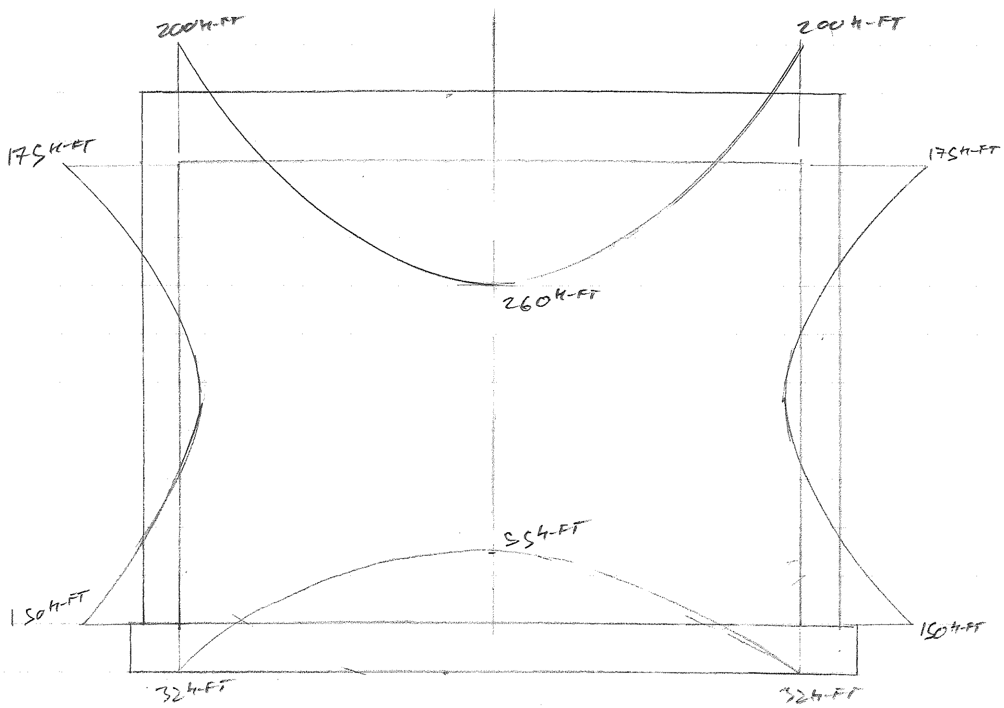


FLSmidth	PROJECT: CC&V High Grade Ore	PROJ NO. 11021
		CALC NO. C-002
	SUBJECT: Reclaim Vault Forces	BY ECS DATE 9-16-11
		CHK DATE
		SHEET OF REV A

* FORCES TAKEN FROM ROBOT MODEL [1.2D+1.6(L+H)]



SHEAR



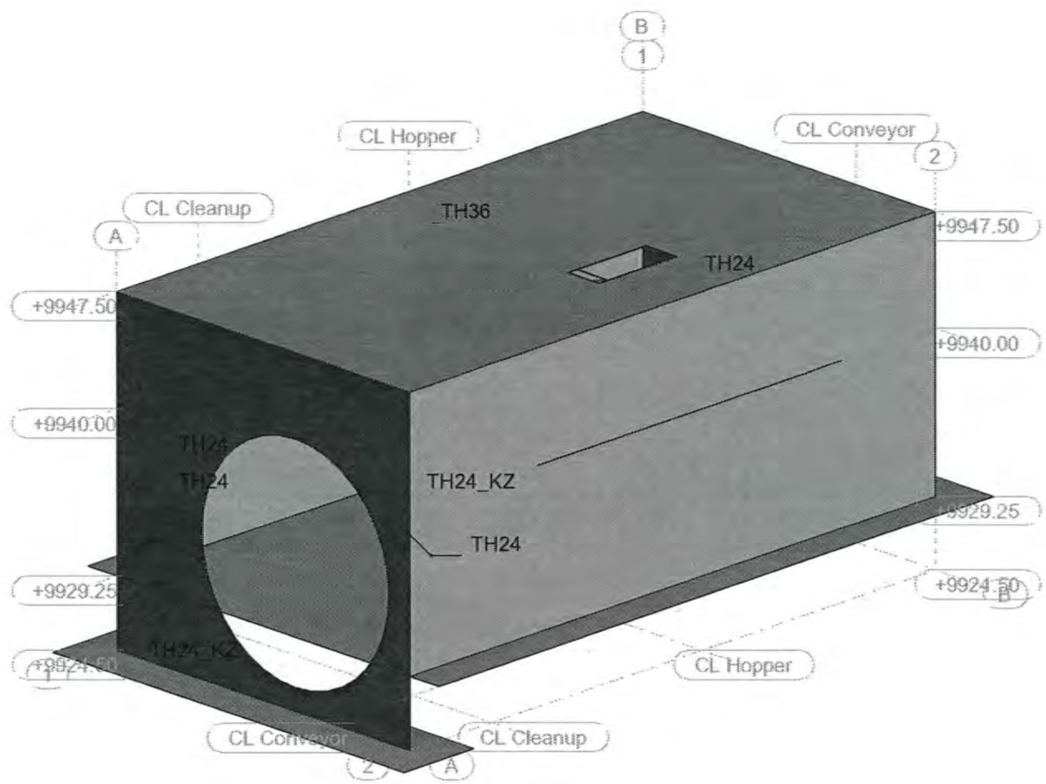
MOMENT

7158 S. FLSmidth Drive
Midvale, UT 84047-5559

Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault Rev A**
Client: **CC&V**

View -

PLATE THICKNESS



Cases: 1 (Self Weight)

Date : 20/09/11
File: **Vault - Rev A.rtd**

Page : **1**
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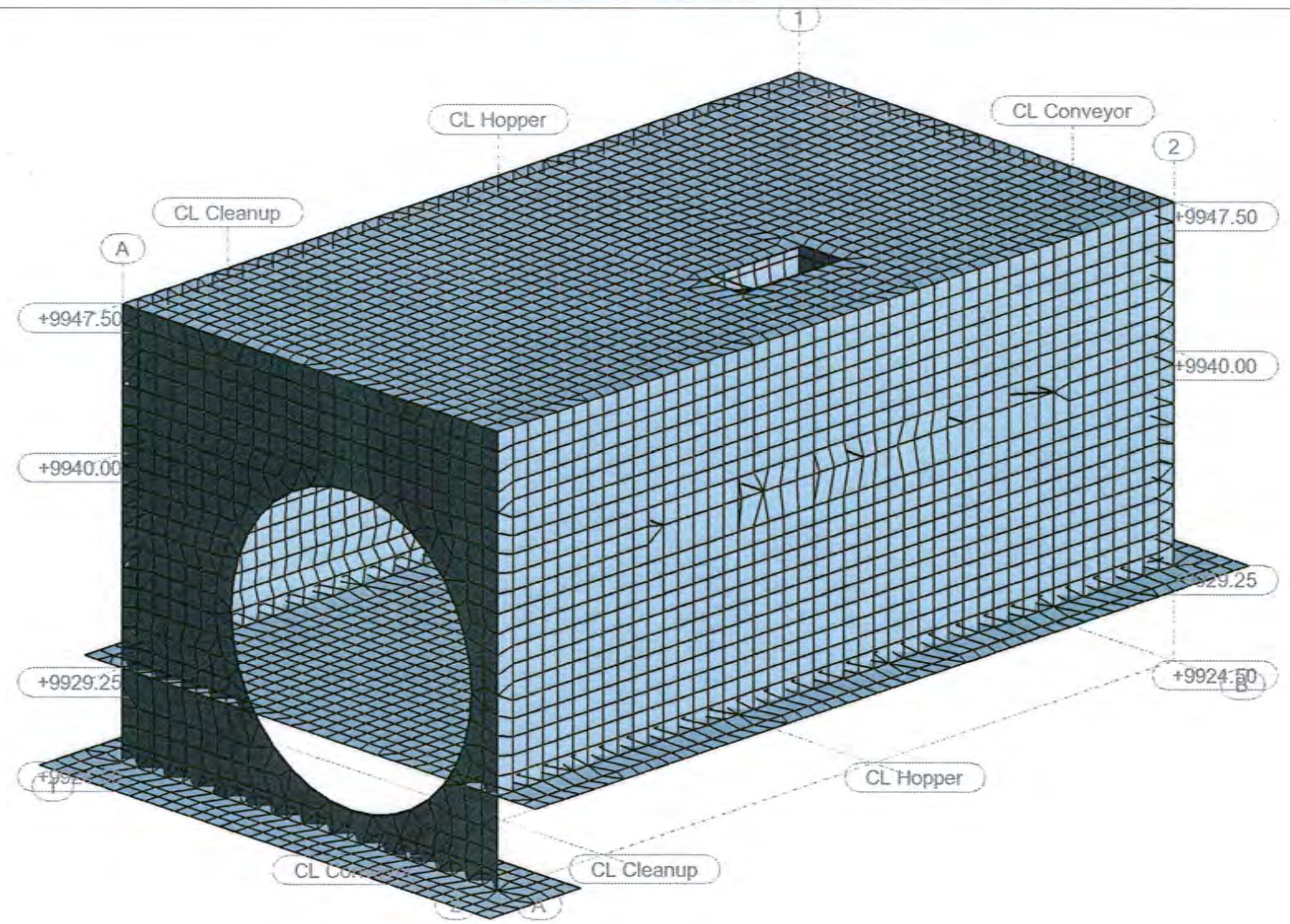
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault Rev A**
Client: **CC&V**

View - Cases: 11 (1.2D+1.6(L+H))

FE MESHING



Date : 20/09/11
File: **Vault - Rev A.rtd**

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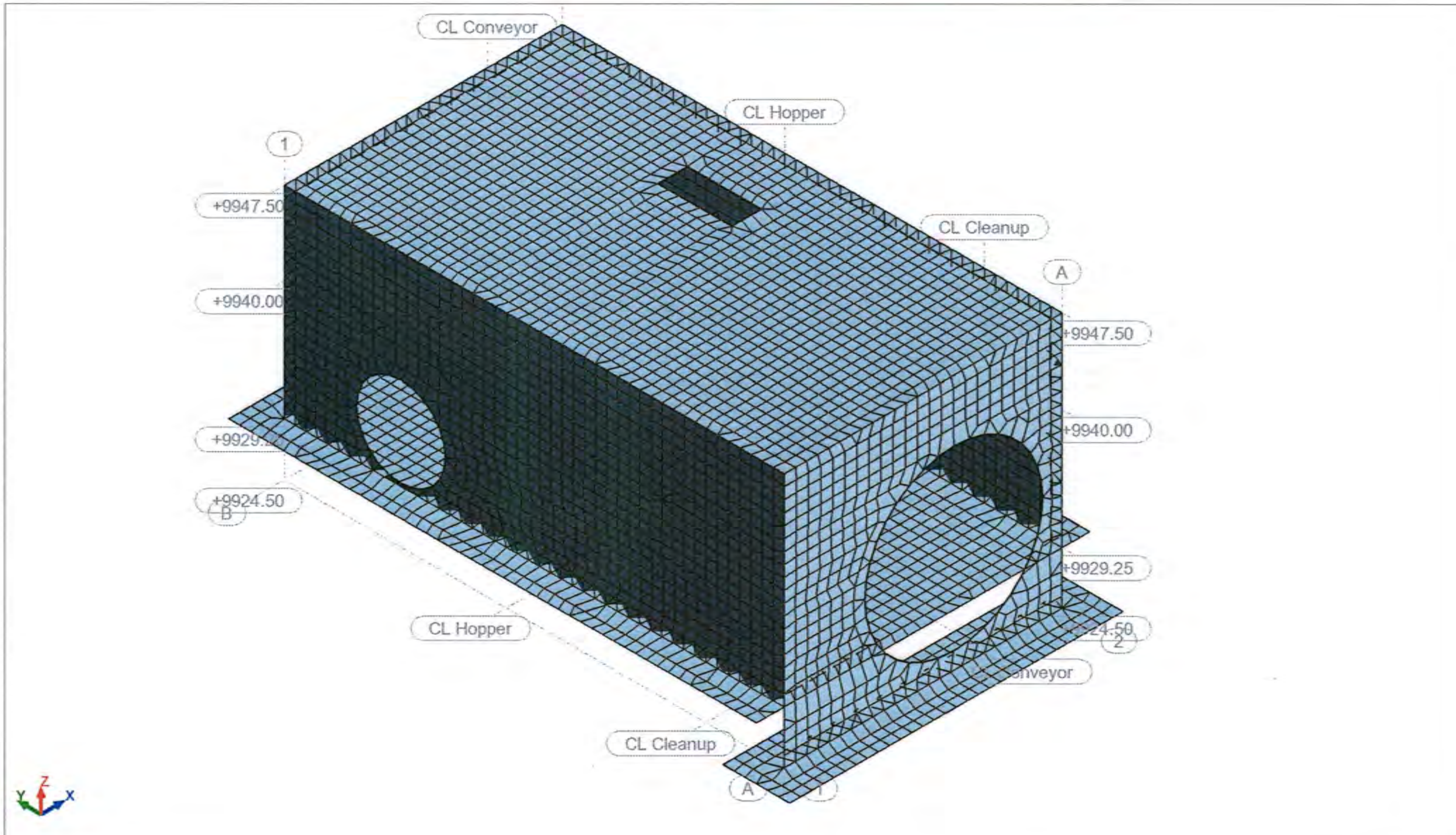
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault Rev A**
Client: **CC&V**

View - Cases: 11 (1.2D+1.6(L+H))

FE MESHING



Date : 20/09/11
File: **Vault - Rev A.rtd**

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Engineer: **Ed Summmers**
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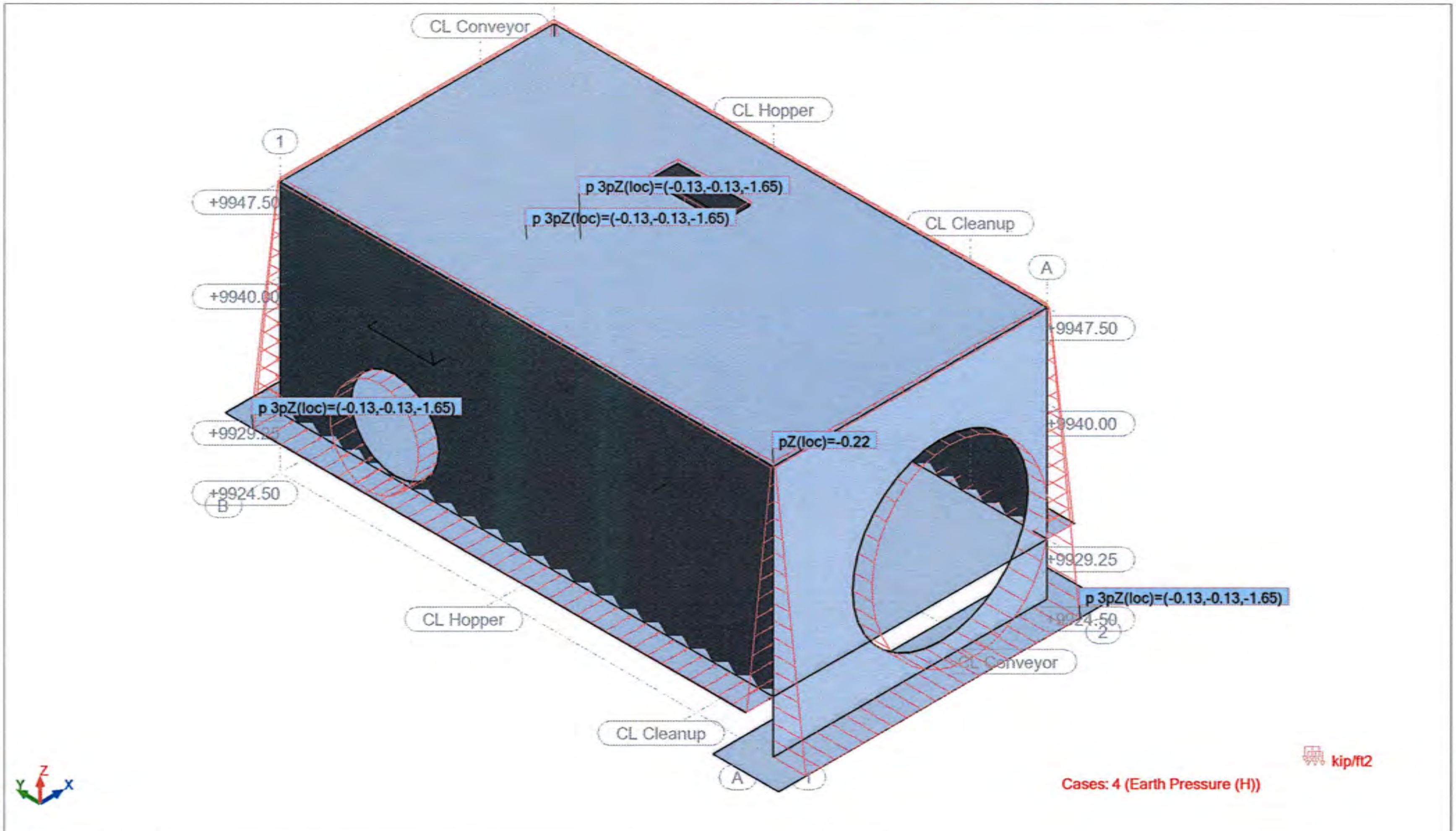
Project: **Vault Rev A**
Client: **CC&V**

View -

APPLIED EARTH PRESSURES



Date : 20/09/11
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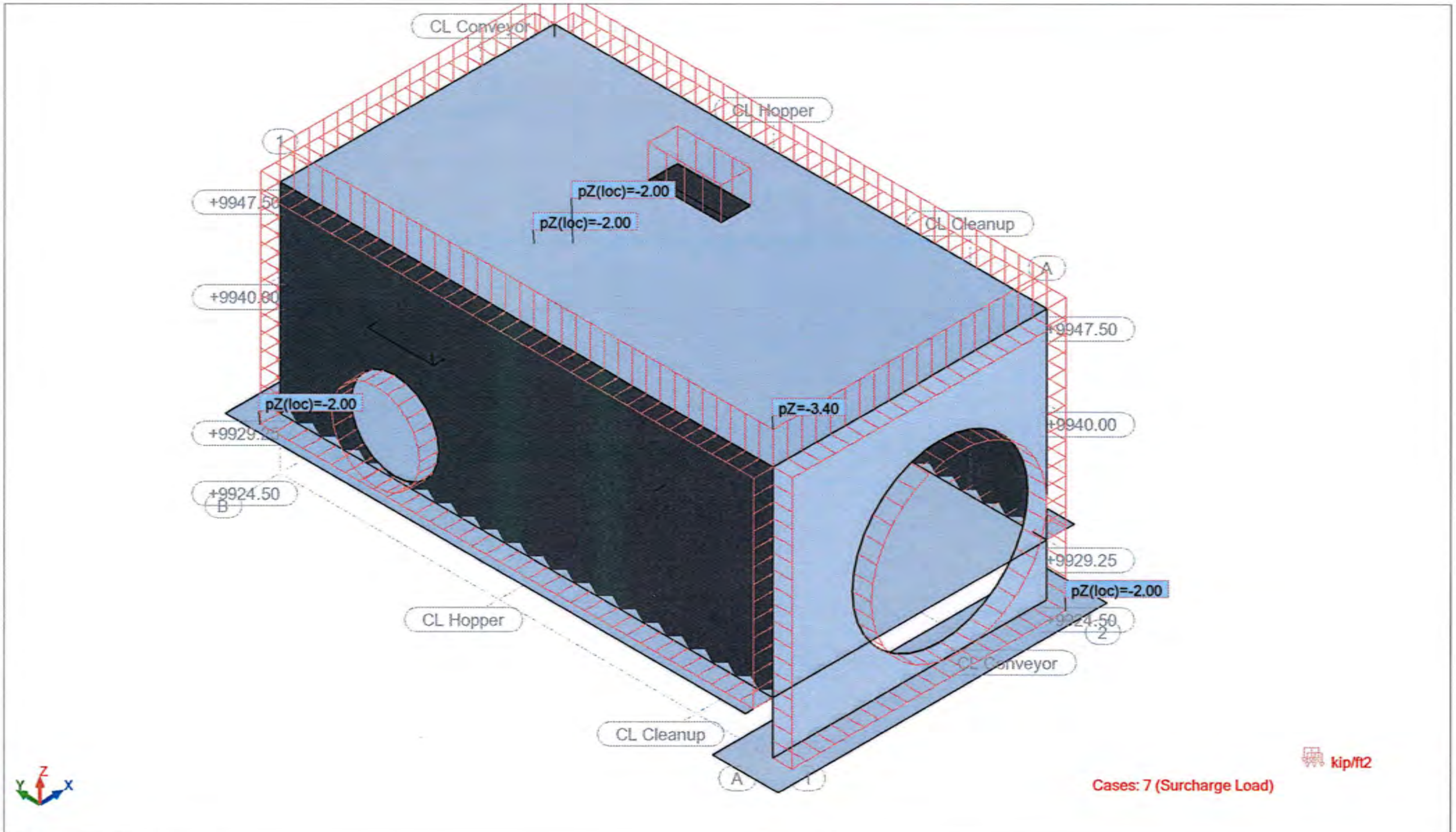
Project: **Vault Rev A**
Client: **CC&V**

View -

SURCHARGE LOAD



Date : 20/09/11
File: **Vault - Rev A.rtd**

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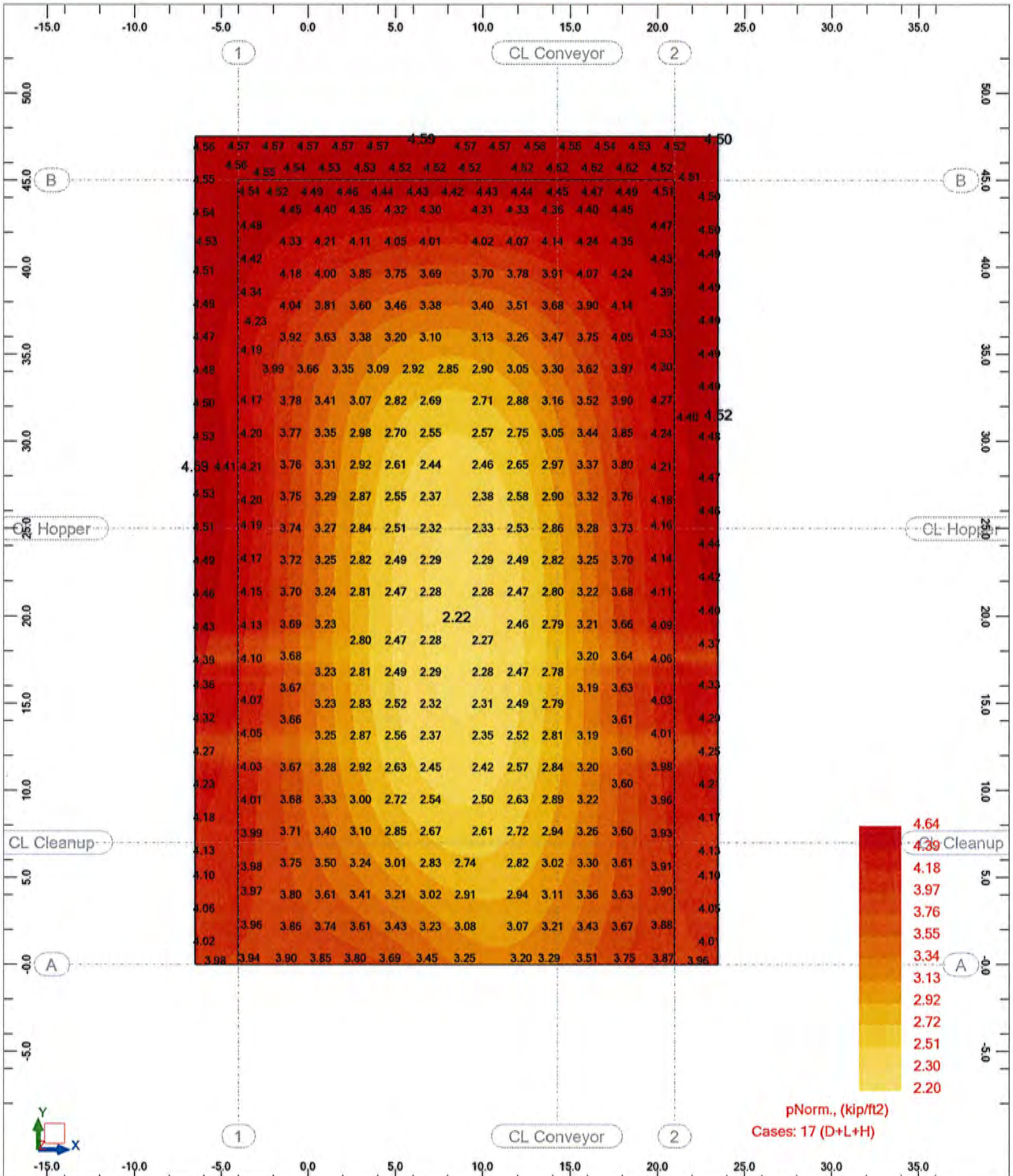
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault Rev A**
Client: **CC&V**

View - pNorm. (kip/ft2) Cases: 17 (D+L+H)

BEARING PRESSURE



Date : 20/09/11
File: **Vault - Rev A.rtd**

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Roof Reinforcement Requirement

per ACI 318-08

Material Properties:

t =	48	in	Total thickness of concrete
f'_c =	4	ksi	Compressive strength of concrete
λ =	1		Lightweight concrete factor
F_y =	60	ksi	Yield stress of reinforcement
ϕ_f =	0.9		Flexural reduction factor
ϕ_v =	0.75		Shear reduction factor

Flexural Requirement:

	X Direction		Y Direction			
	Top	Bottom	Top	Bottom*		
Mu =	226	250	100	120	kip*ft	Factored maximum moment
c =	2	2	2	2	in	Reinforcement cover
Bar:	# 10	# 10	# 8	# 8		Reinforcement provided
b =	12	12	12	12	in	Bar spacing
A_s =	1.27	1.27	0.79	0.79	in^2/ft	Provided steel reinforcement
d =	46	46	46	46	in	Distance to reinforcement
M_n =	286.17	286.17	179.41	179.41	k-ft	Nominal moment capacity
ϕM_n =	257.55	257.55	161.46	161.46	k-ft	Factored moment capacity
Ratio =	**0.88**	**0.97**	**0.62**	**0.74**		

Shear Requirement

	X Dir.	Y Dir.		
V_u =	50	38	kip/ft	Factored maximum shear
V_n =	69.8	69.8	kip/ft	Nominal shear capacity per ft widtl
ϕV_n =	52.37	52.37	kip/ft	Factored shear capacity per ft widt
Ratio =	**0.95**	**0.73**		

Temperature Requirement

ρ =	0.0018	0.0018		Required percentage for temperature steel
ρ_w =	0.0044	0.0027	%	Reinforcement provided
$A_{s,Required}$ =	1.0368	1.0368	in^2	

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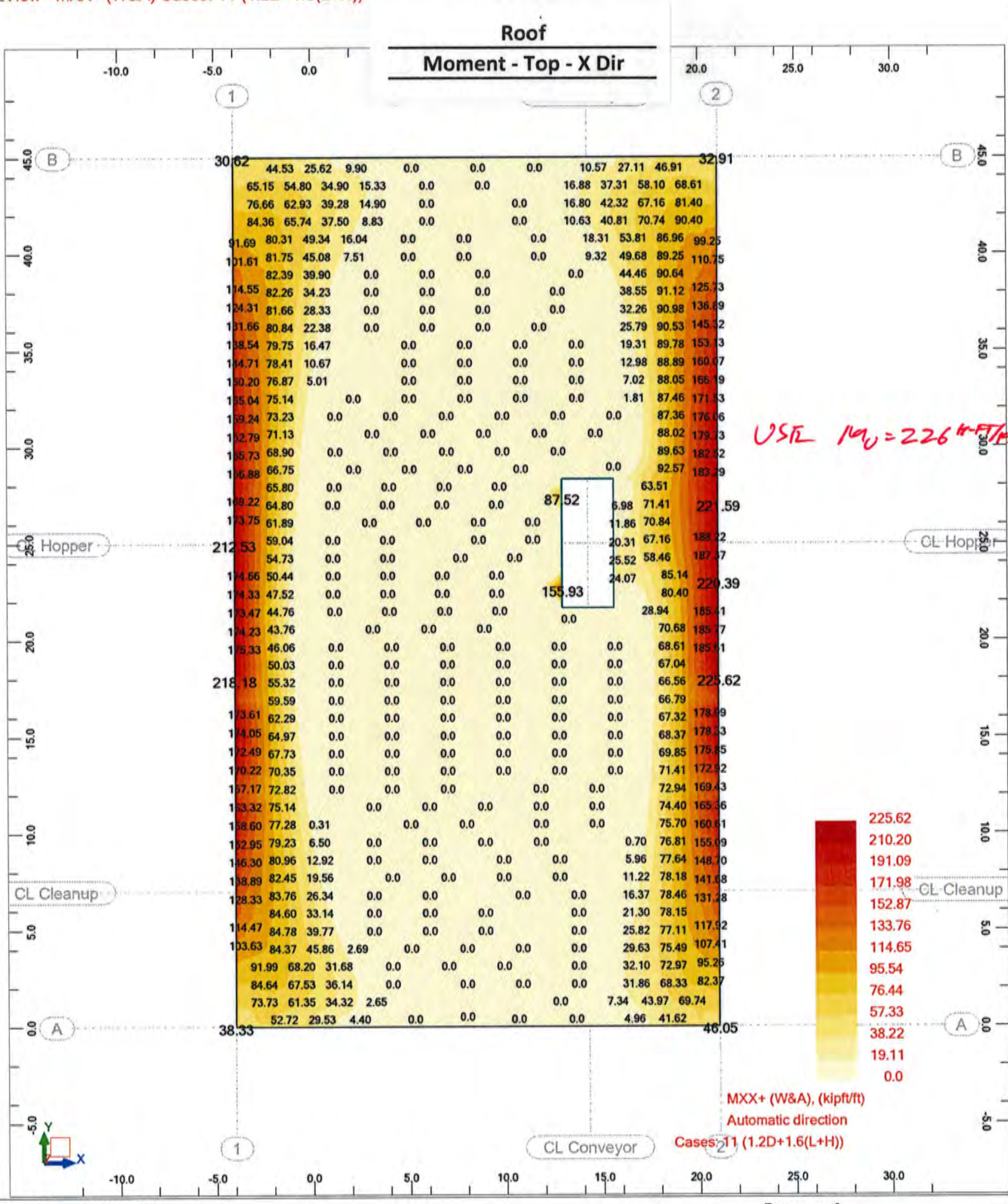
7158 S. FLSmidth Drive
Midvale, UT 84047-5559

Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX+ (W&A) Cases: 11 (1.2D+1.6(L+H))



Date : 30/09/11
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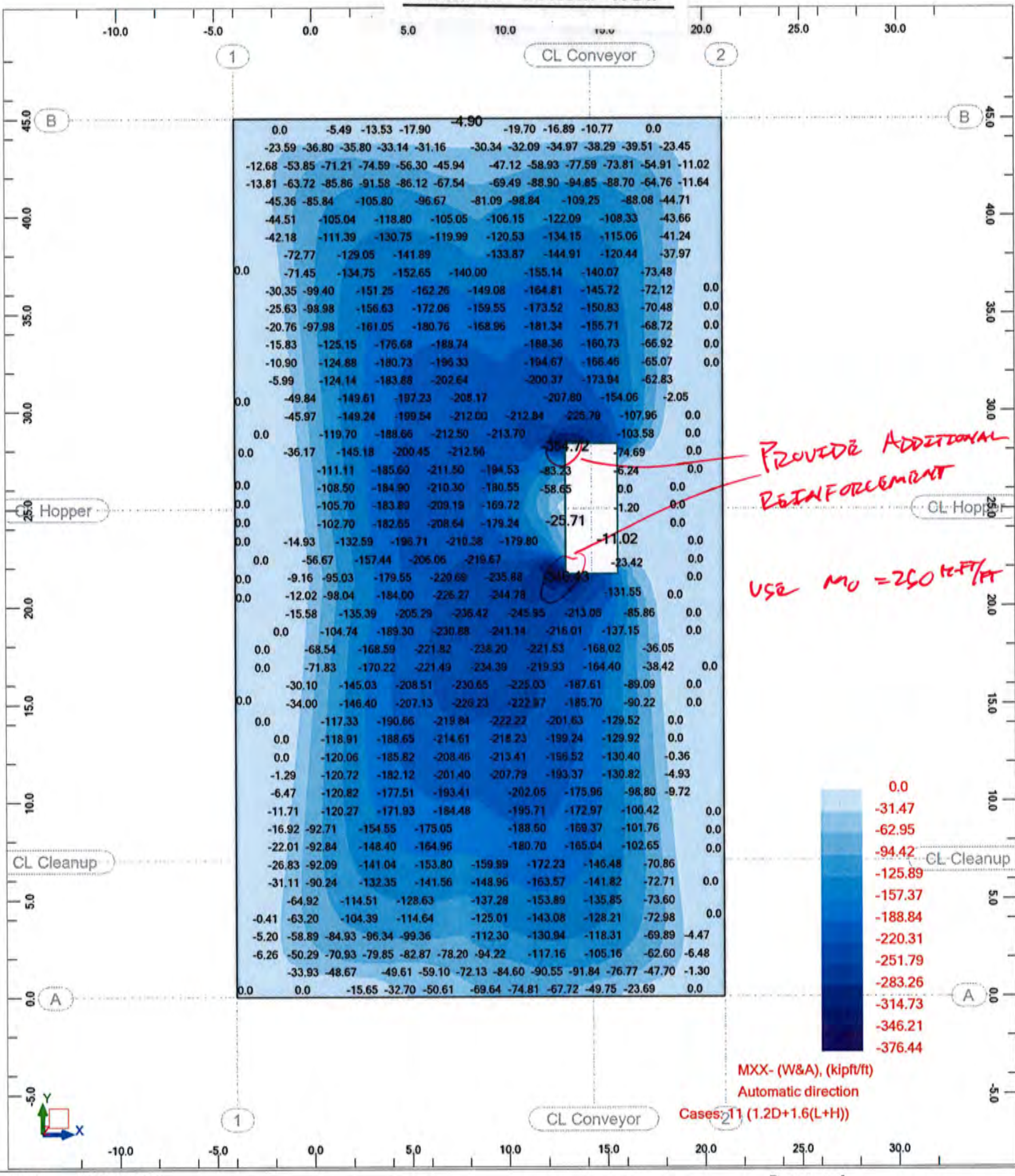
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX- (W&A) Cases: 11 (1.2D+1.6(L+H))

Roof

Moment - Bottom - X Dir



PROVIDE ADDITIONAL REINFORCEMENT

USE $M_U = 260$ ft-ft/ft

Date : 30/09/11
File: **Vault - Rev C.rtd**

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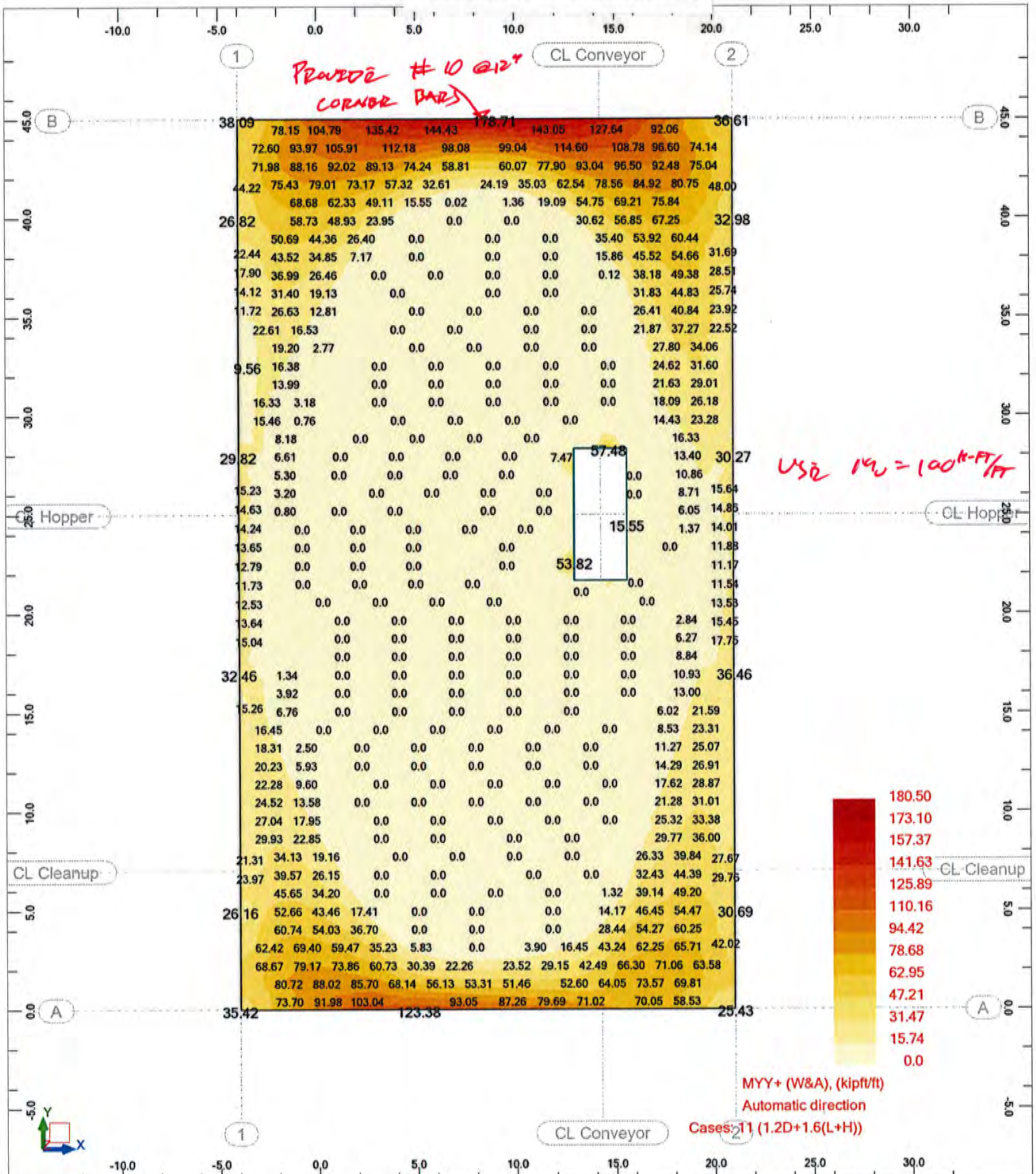
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MYY+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Roof

Moment - Top - Y Dir



Date : 30/09/11
File: **Vault - Rev C.rtd**

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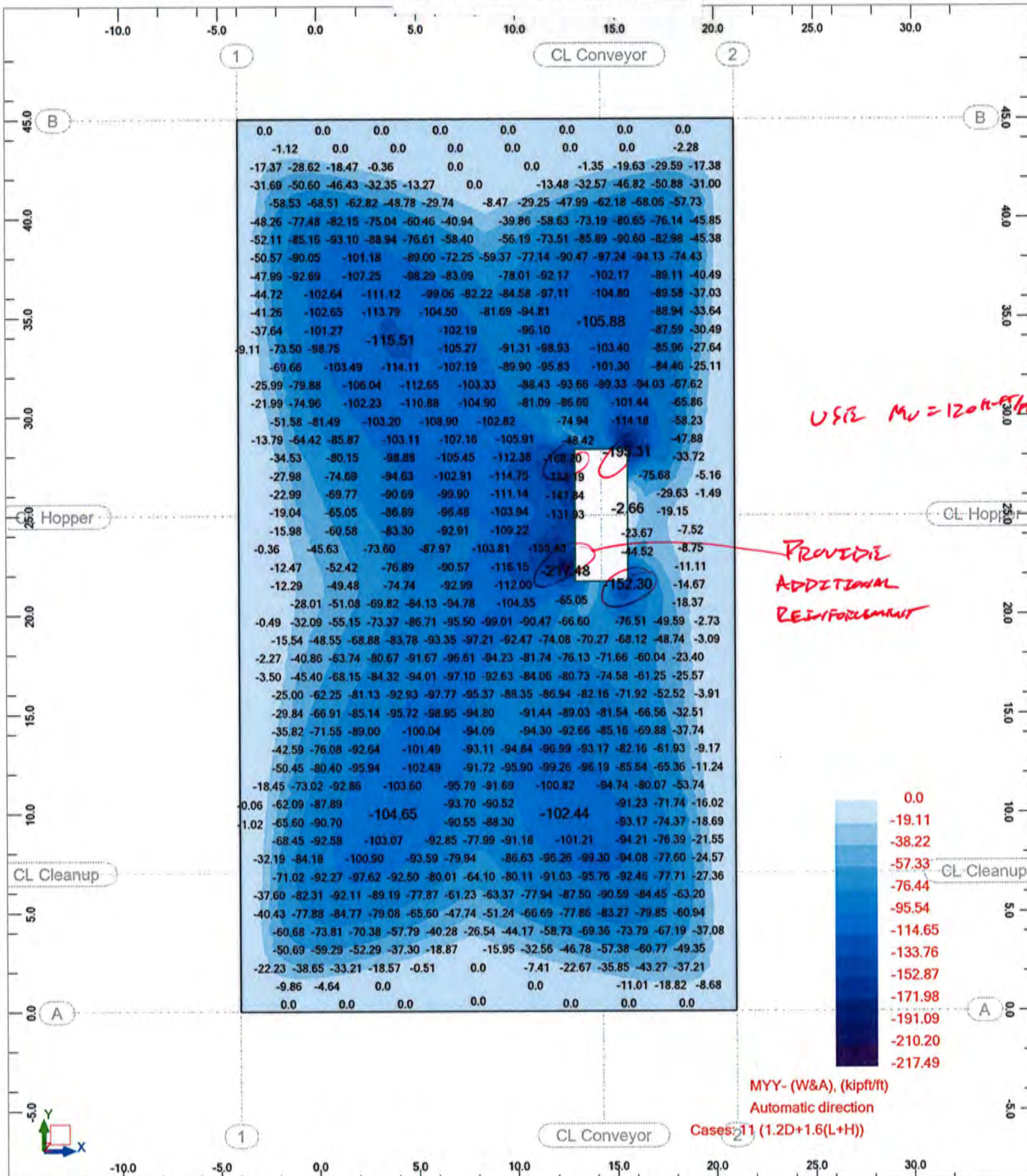
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MYY- (W&A) Cases: 11 (1.2D+1.6(L+H))

Roof

Moment - Bottom - Y Dir



Date : 30/09/11
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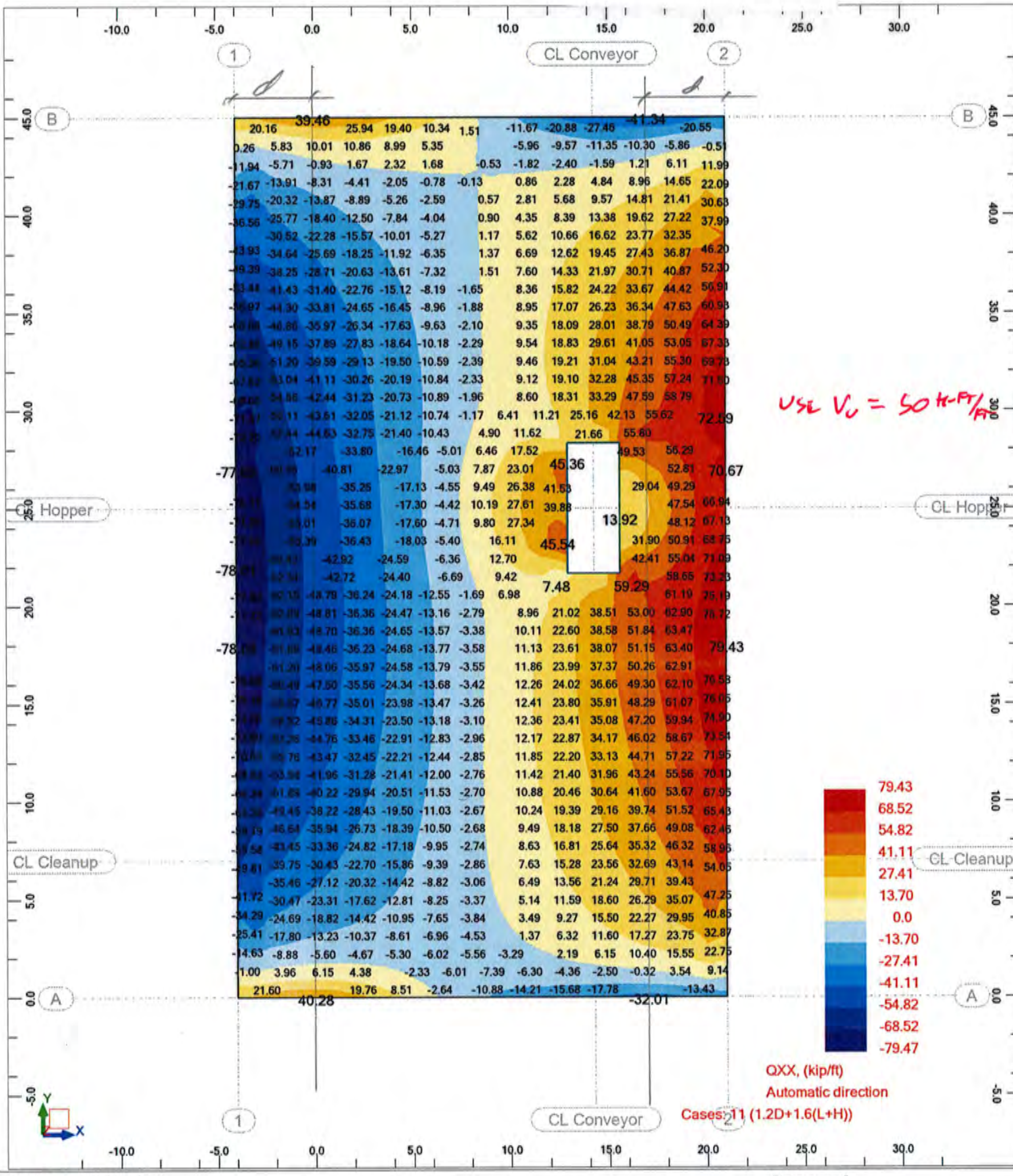
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - QXX (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Roof

Shear - X Dir



Date : 30/09/11
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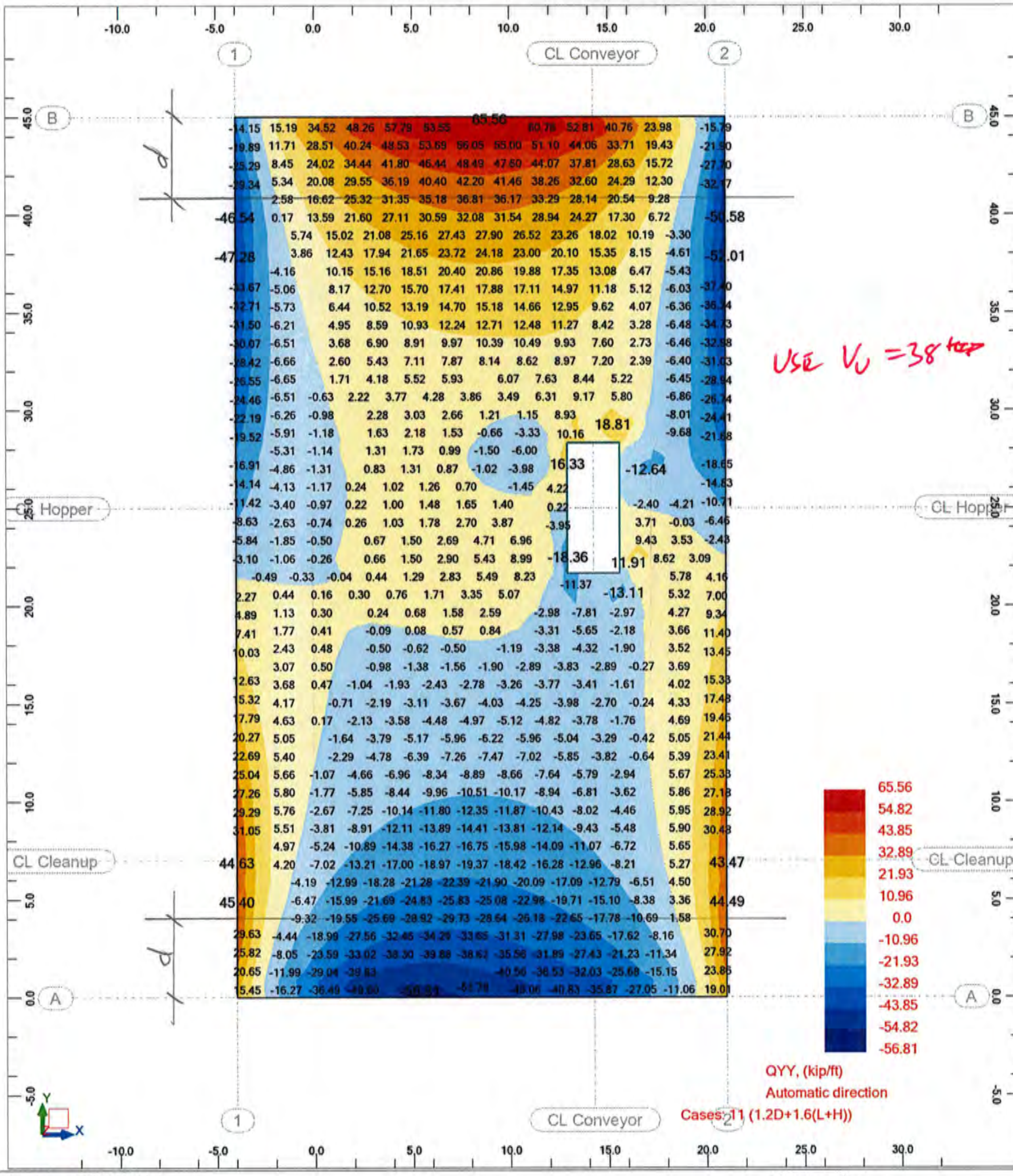
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - QYY (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Roof

Shear - Y Dir



Date : 30/09/11
File: **Vault - Rev C.rtd**

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Mat Reinforcement Requirement

per ACI 318-08

Material Properties:

t =	36	in	Total thickness of concrete
f'_c =	4	ksi	Compressive strength of concrete
λ =	1		Lightweight concrete factor
F_y =	60	ksi	Yield stress of reinforcement
ϕ_f =	0.9		Flexural reduction factor
ϕ_v =	0.75		Shear reduction factor

Flexural Requirement:

	X Direction		Y Direction			
	Top	Bottom	Top	Bottom*		
Mu =	150	185	83	100	kip*ft	Factored maximum moment
c =	2	2	2	2	in	Reinforcement cover
Bar:	# 10	# 10	# 8	# 8		Reinforcement provided
b =	12	12	12	12	in	Bar spacing
A_s =	1.27	1.27	0.79	0.79	in^2/ft	Provided steel reinforcement
d =	34	34	34	34	in	Distance to reinforcement
M_n =	209.97	209.97	132.01	132.01	k-ft	Nominal moment capacity
ϕM_n =	188.97	188.97	118.80	118.80	k-ft	Factored moment capacity
Ratio =	**0.79**	**0.98**	**0.70**	**0.84**		

Shear Requirement

	X Dir.	Y Dir.		
V_u =	38	38	kip/ft	Factored maximum shear
V_n =	51.6	51.6	kip/ft	Nominal shear capacity per ft widtl
ϕV_n =	38.71	38.71	kip/ft	Factored shear capacity per ft wid
Ratio =	**0.98**	**0.98**		

Temperature Requirement

ρ =	0.0018	0.0018		Required percentage for temperature steel
ρ_w =	0.0059	0.0037	%	Reinforcement provided
$A_{s,Required}$ =	0.7776	0.7776	in^2	
	OK	**OK**		

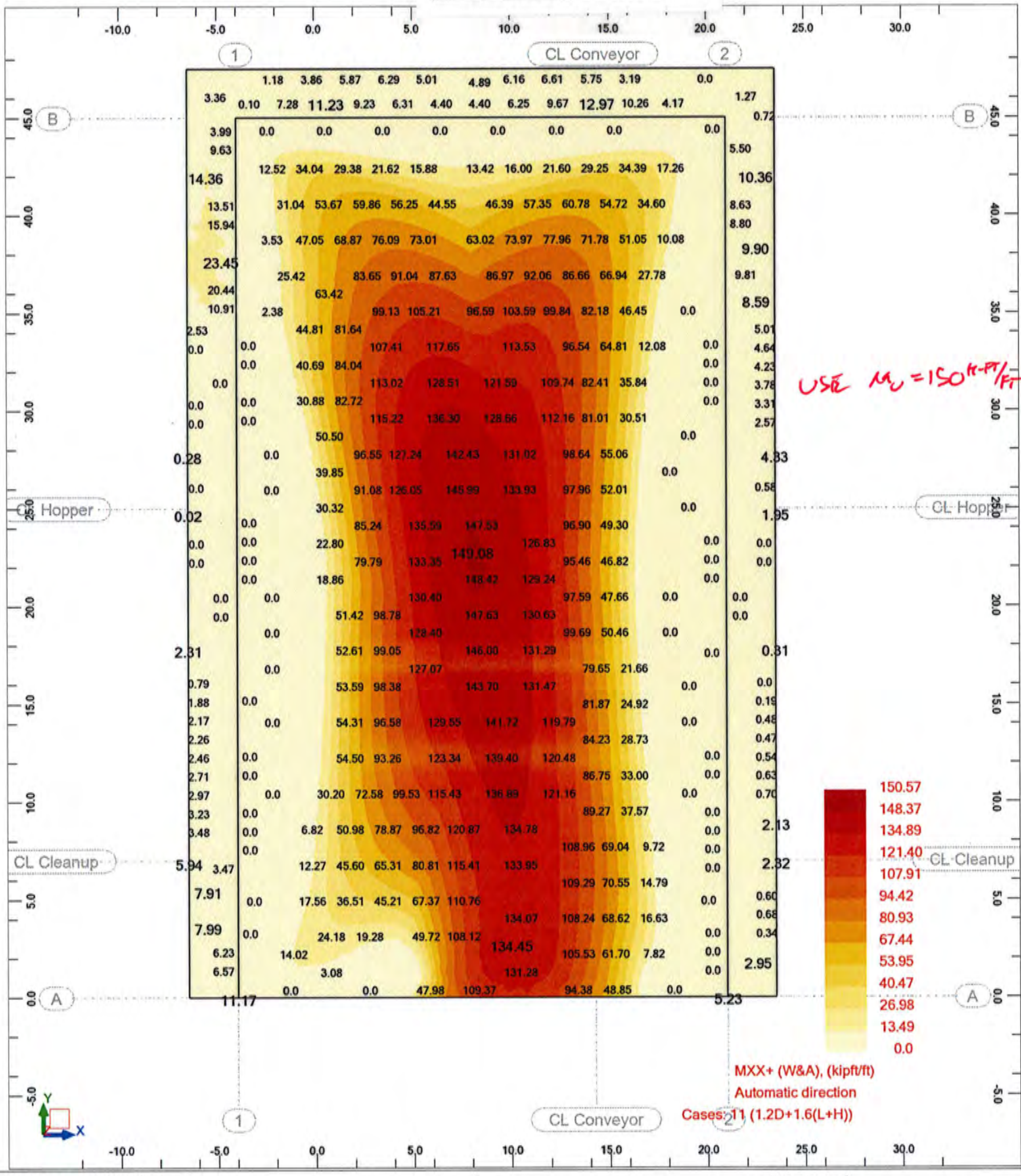
FLSmidth
7158 S. FLSmidth Drive
Midvale, UT 84047-5559

Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Mat

Moment - Top - X Dir



USE $M_u = 150^{K\text{-}FT}/_{FT}$

Date : 30/09/11
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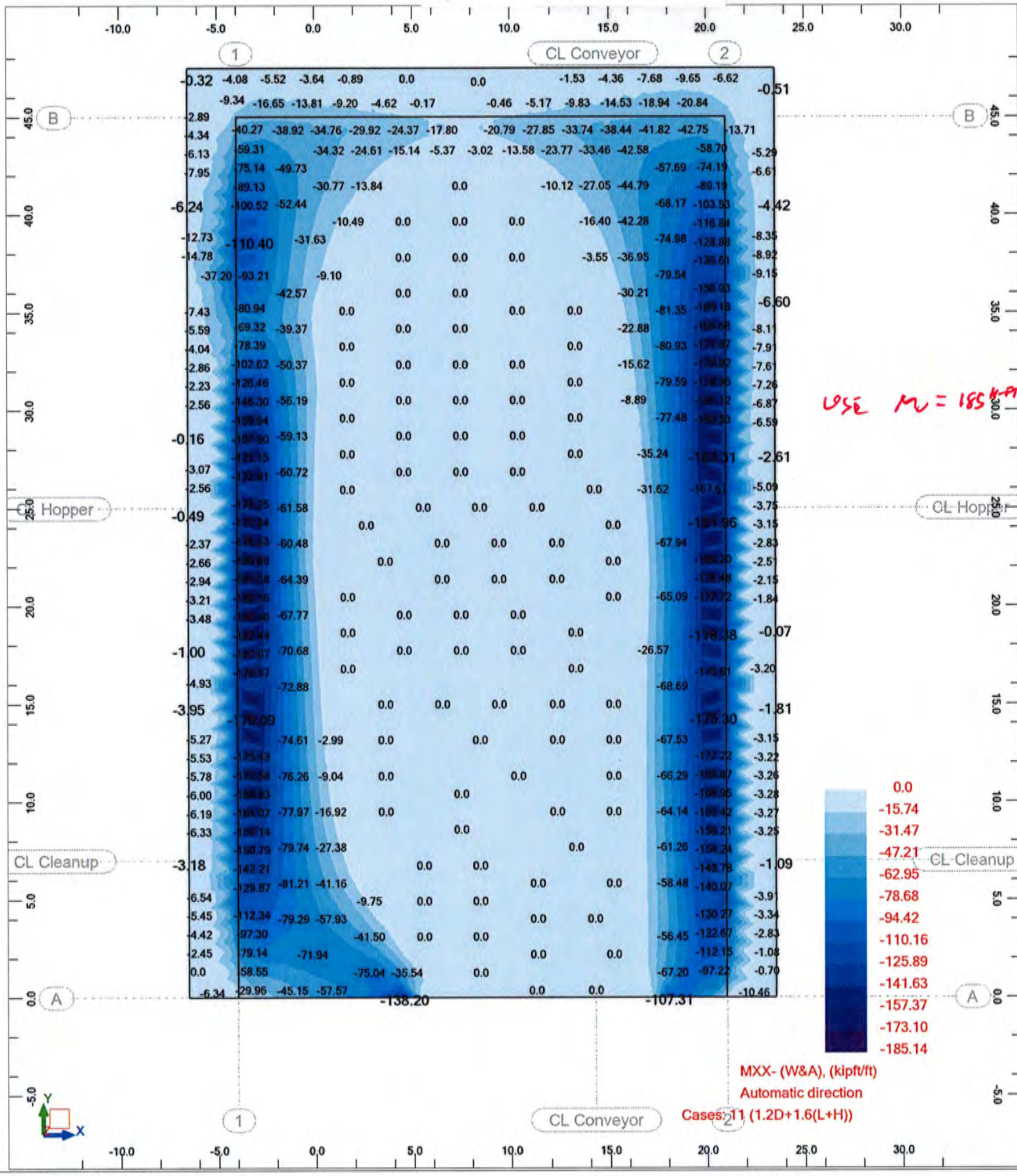
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX- (W&A) Cases: 11 (1.2D+1.6(L+H))

Mat

Moment - Bottom - X Dir



USE M = 185 k-ft/ft

Date : 30/09/11
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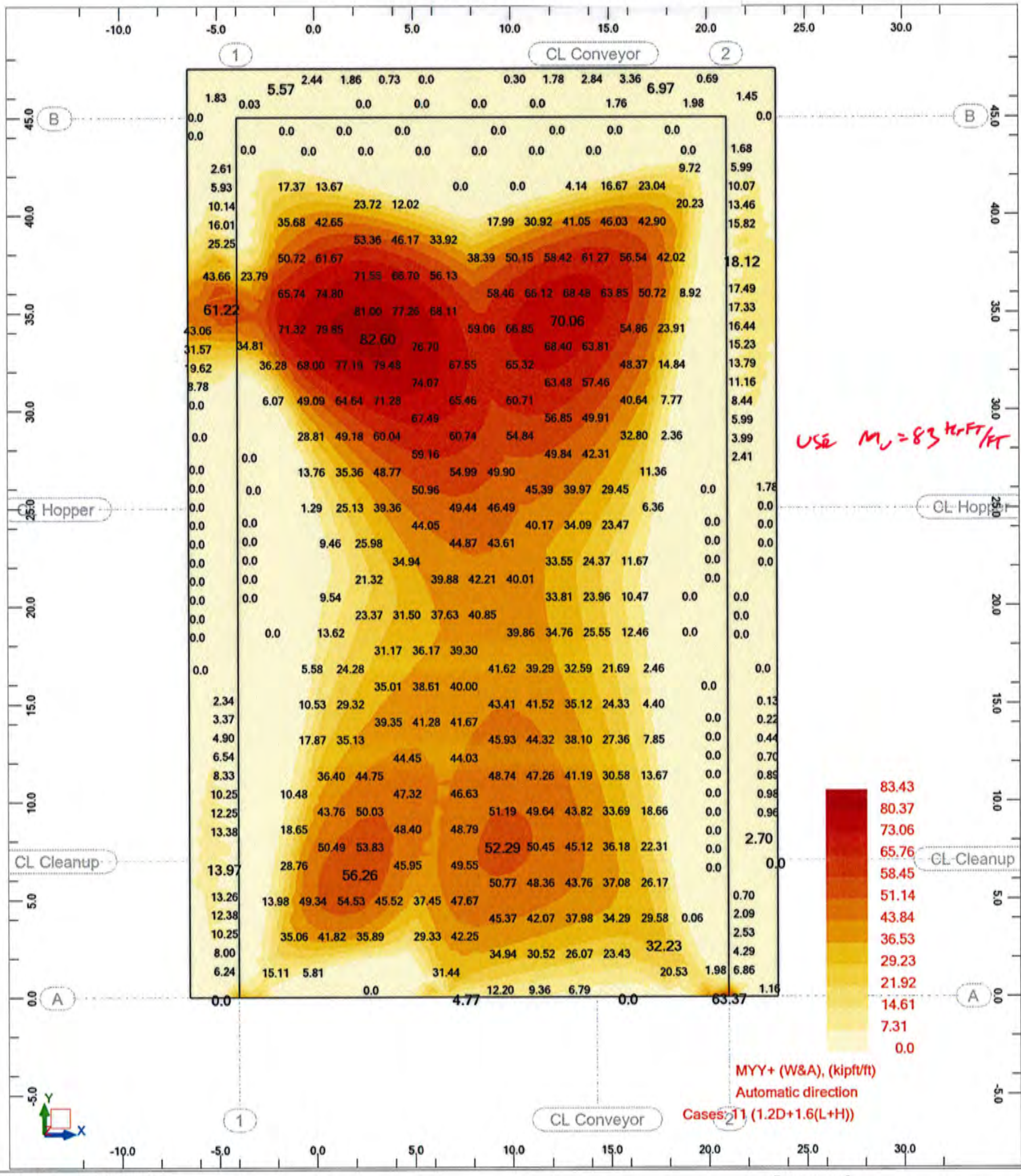
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MYY+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Mat

Moment - Top - Y Dir



Date : 30/09/11
File: **Vault - Rev C.rtd**

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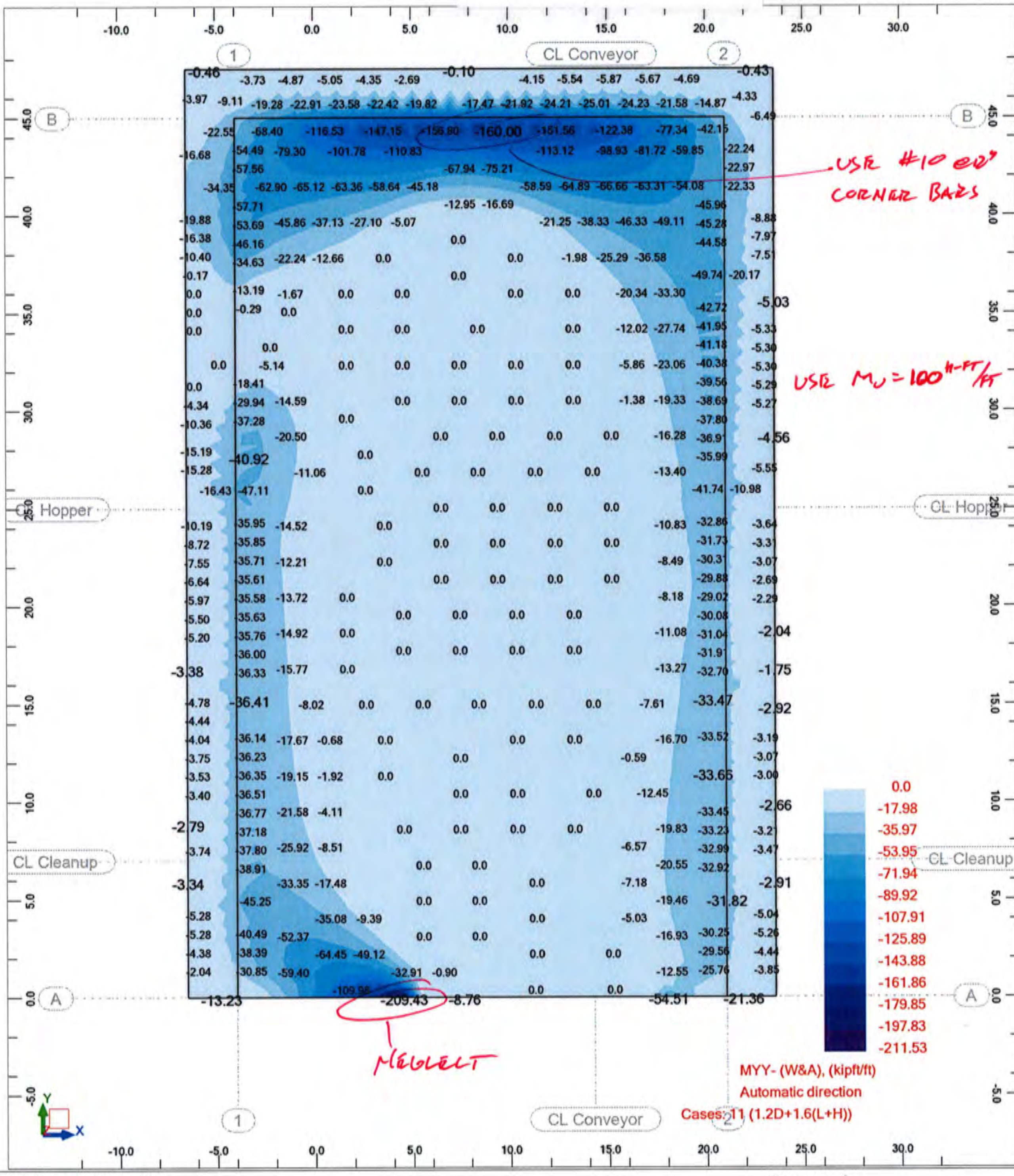
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MYY- (W&A) Cases: 11 (1.2D+1.6(L+H))

Mat

Moment - Bottom - Y Dir



Date : 30/09/11
File: **Vault - Rev C.rtd**

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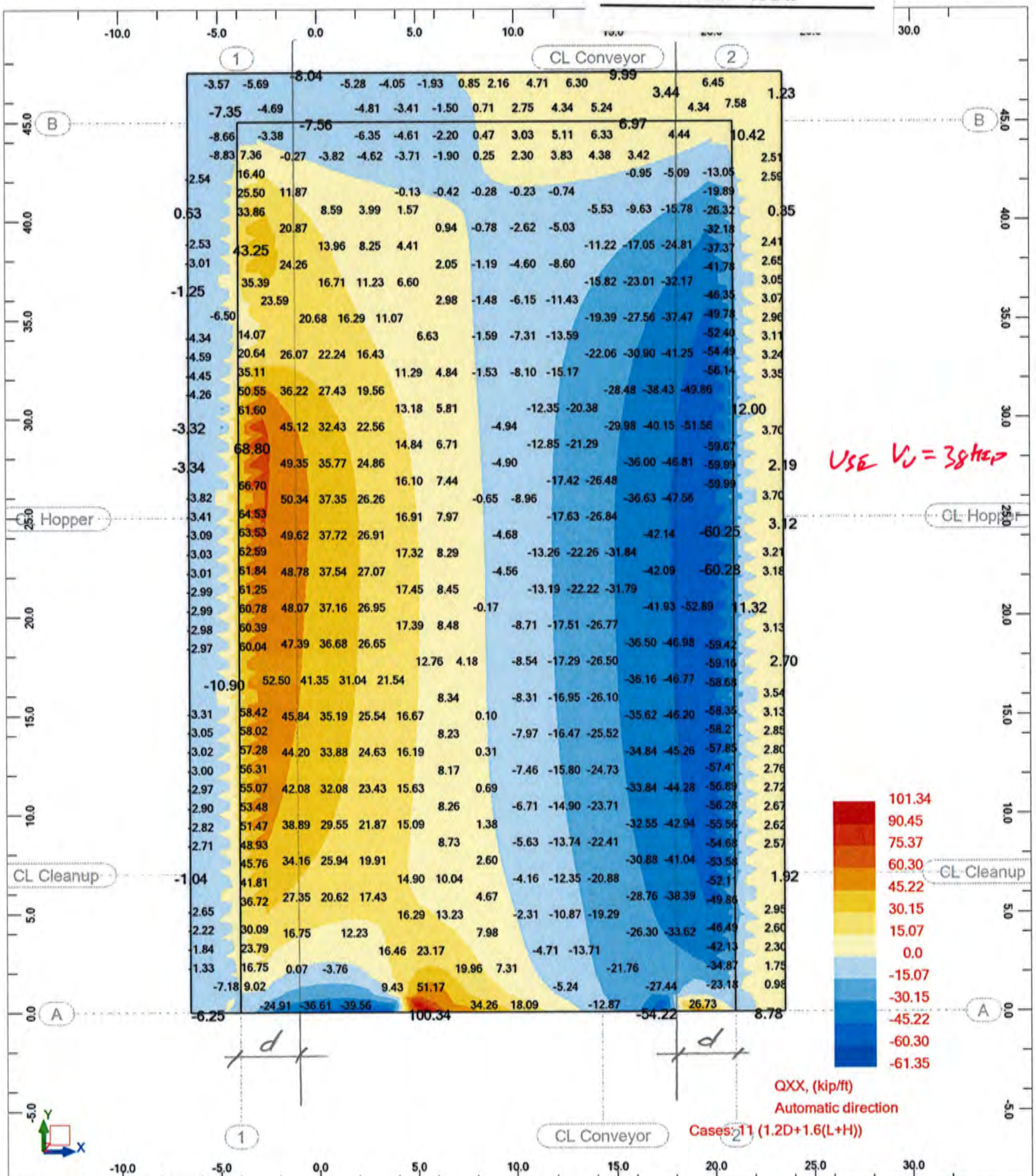
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - QXX (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Mat

Shear - X Dir



USE $V_u = 38$ kip

Date : 30/09/11
File: **Vault - Rev C.rtd**

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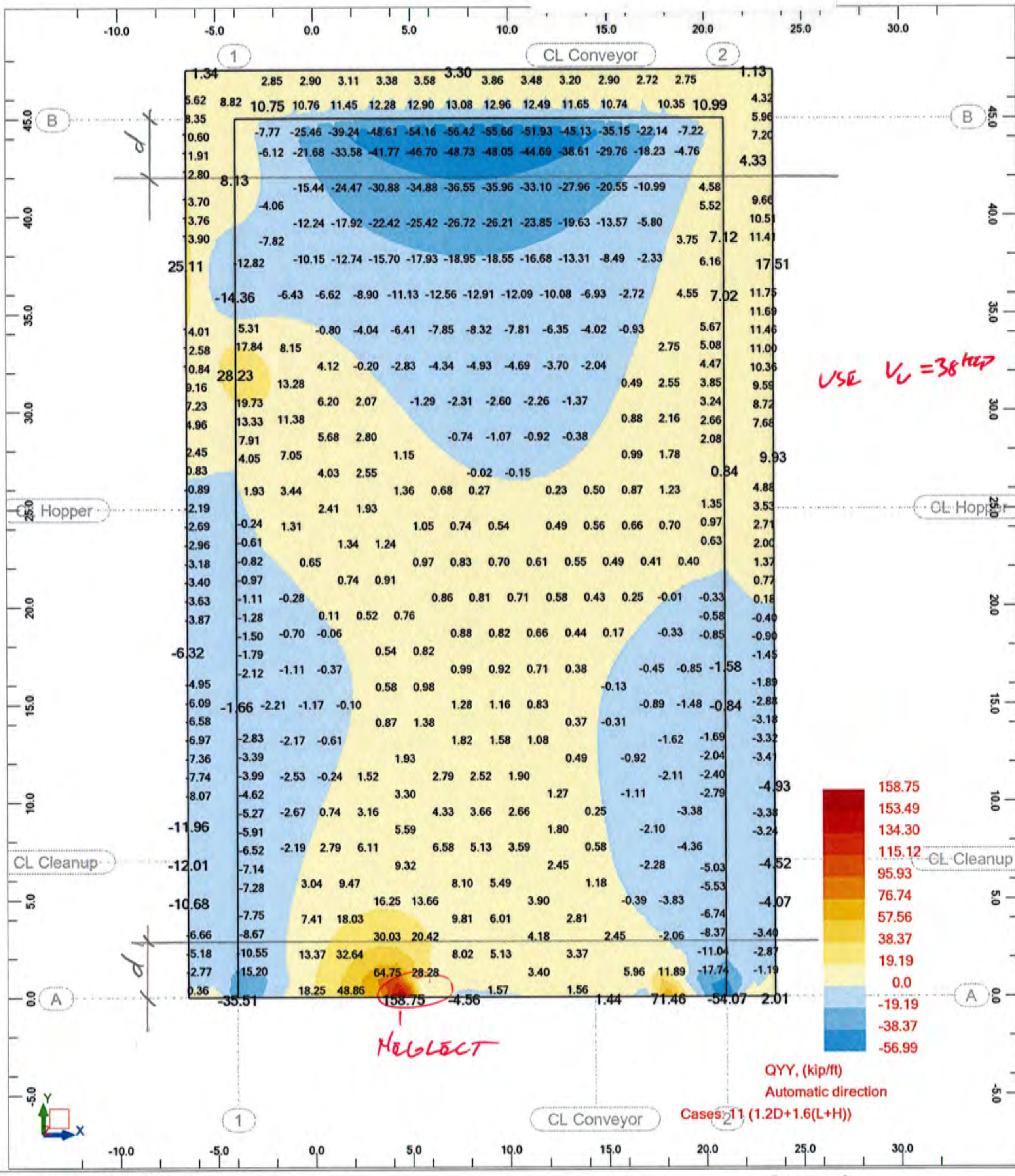
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - QYY (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Mat

Shear - Y Dir



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Back Wall Reinforcement Requirement

per ACI 318-08

Material Properties:

t =	36	in	Total thickness of concrete
f'_c =	4	ksi	Compressive strength of concrete
λ =	1		Lightweight concrete factor
F_y =	60	ksi	Yield stress of reinforcement
ϕ_f =	0.9		Flexural reduction factor
ϕ_v =	0.75		Shear reduction factor

Flexural Requirement:

	X Direction		Y Direction			
	Outside	Inside	Outside*	Inside		
Mu =	68	27	110	49	kip*ft	Factored maximum moment
c =	2	2	2	2	in	Reinforcement cover
Bar:	# 8	# 8	# 8	# 8		Reinforcement provided
b =	12	12	12	12	in	Bar spacing
A_s =	0.79	0.79	0.79	0.79	in^2/ft	Provided steel reinforcement
d =	34	34	34	34	in	Distance to reinforcement
M_n =	132.01	132.01	132.01	132.01	k-ft	Nominal moment capacity
ϕM_n =	118.80	118.80	118.80	118.80	k-ft	Factored moment capacity
Ratio =	**0.57**	**0.23**	**0.93**	**0.41**		

Shear Requirement

	X Dir.	Y Dir.		
V_u =	30	30	kip/ft	Factored maximum shear
V_n =	51.6	51.6	kip/ft	Nominal shear capacity per ft widtl
ϕV_n =	38.71	38.71	kip/ft	Factored shear capacity per ft wid
Ratio =	**0.78**	**0.78**		

Temperature Requirement

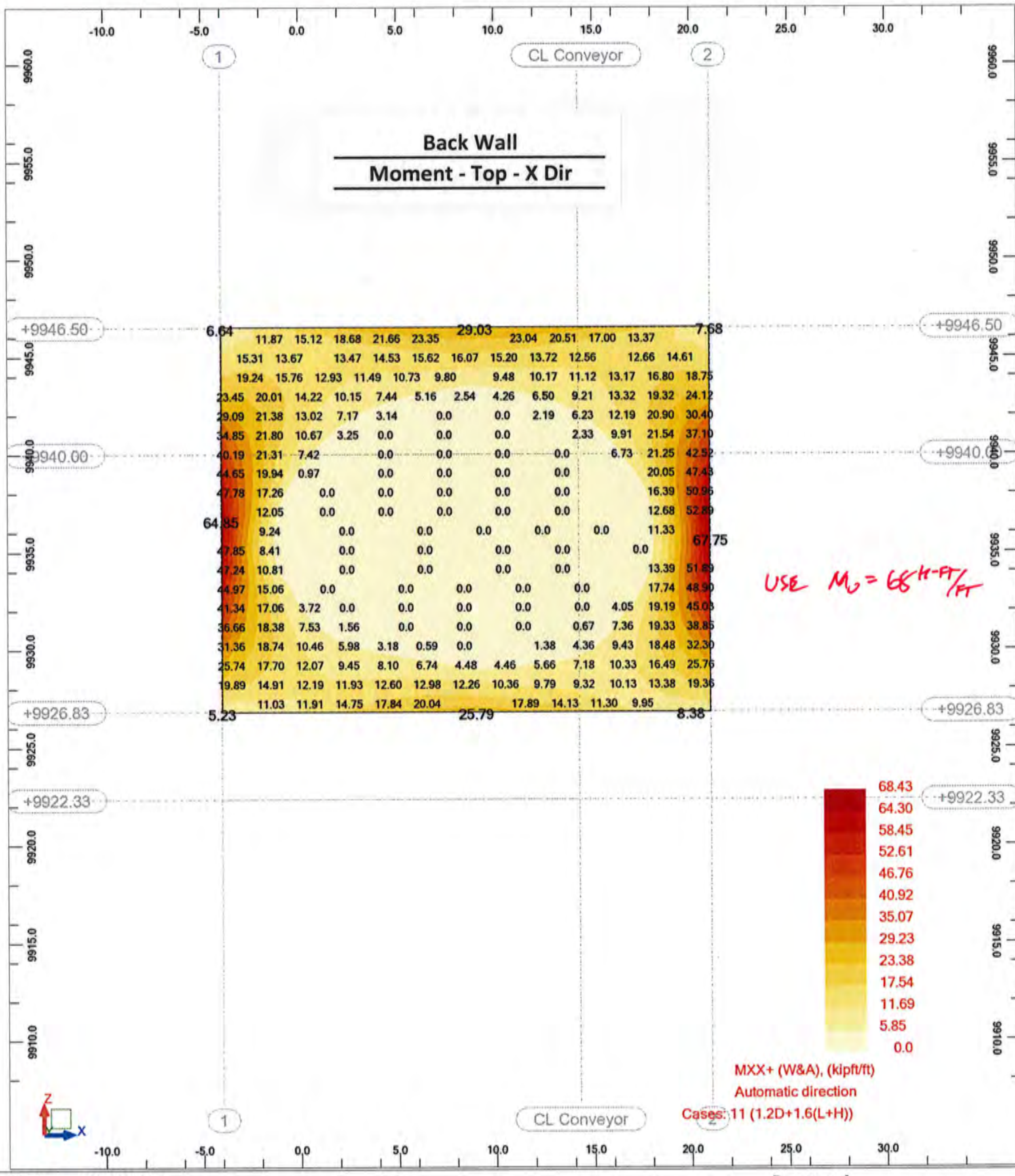
ρ =	0.0018	0.0018		Required percentage for temperature steel
ρ_w =	0.0037	0.0037	%	Reinforcement provided
$A_{s,Required}$ =	0.7776	0.7776	in^2	
	OK	**OK**		

FLSMIDTH

7158 S. FLSmidth Drive
Midvale, UT 84047-5559

Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX+ (W&A) Cases: 11 (1.2D+1.6(L+H))



USE $M_u = 68^{k\text{-}FT}/_{FT}$

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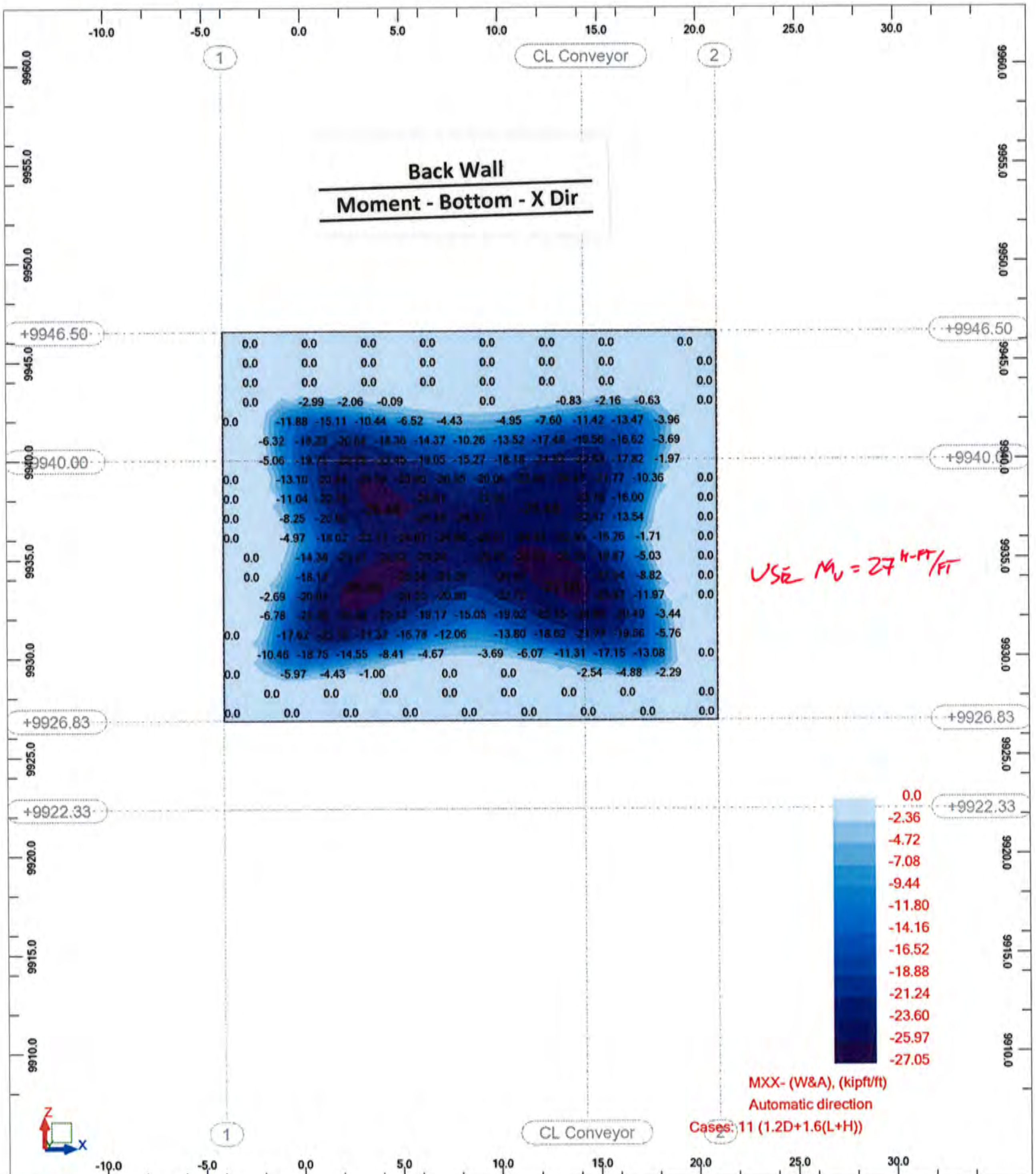
View - MXX- (W&A) Cases: 11 (1.2D+1.6(L+H))



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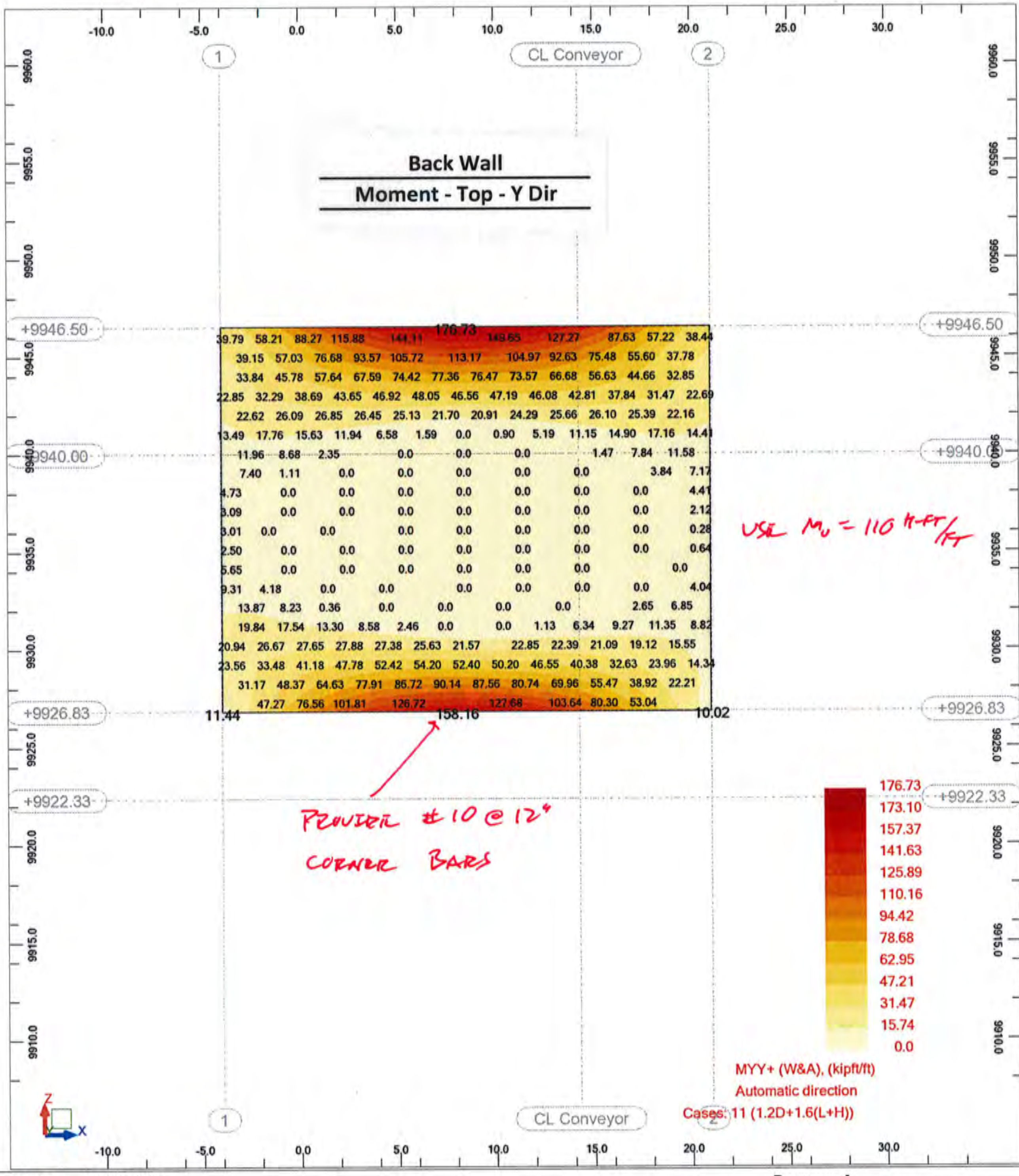
Client: **CC&V**

View - MYY+ (W&A) Cases: 11 (1.2D+1.6(L+H))



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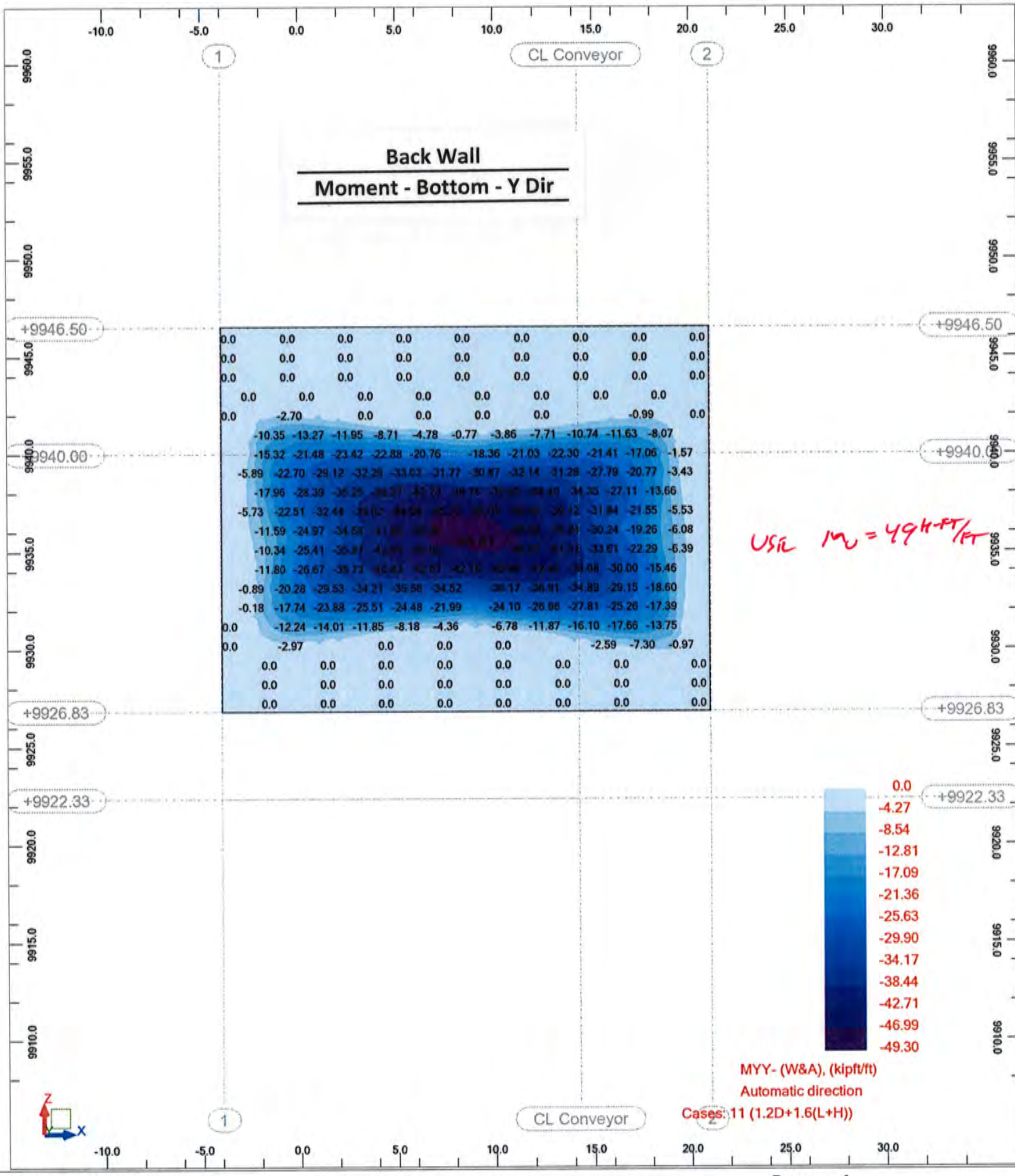
Project: **Vault - Rev C**
Client: **CC&V**

View - MYY- (W&A) Cases: 11 (1.2D+1.6(L+H))



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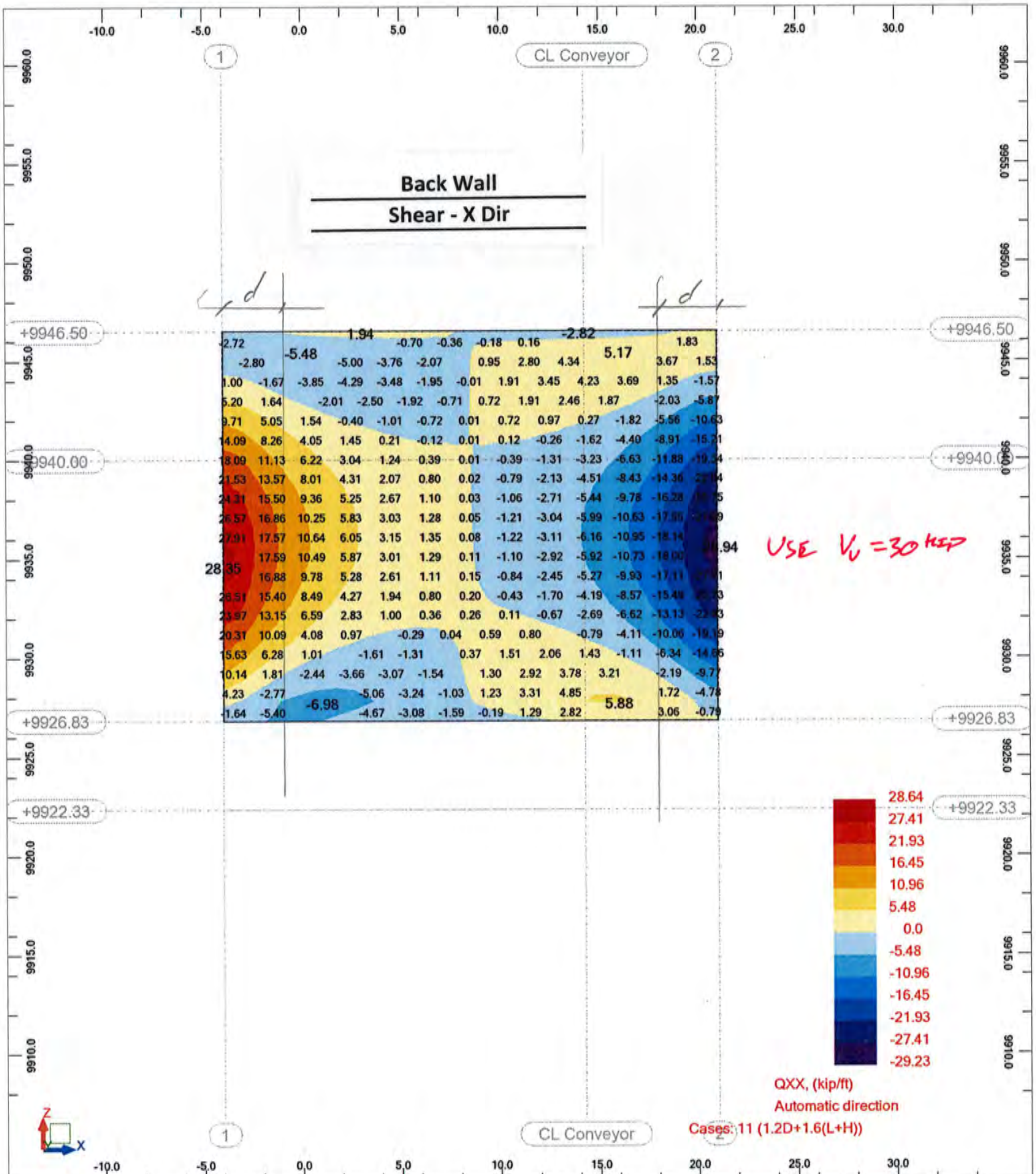
View - QXX (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))



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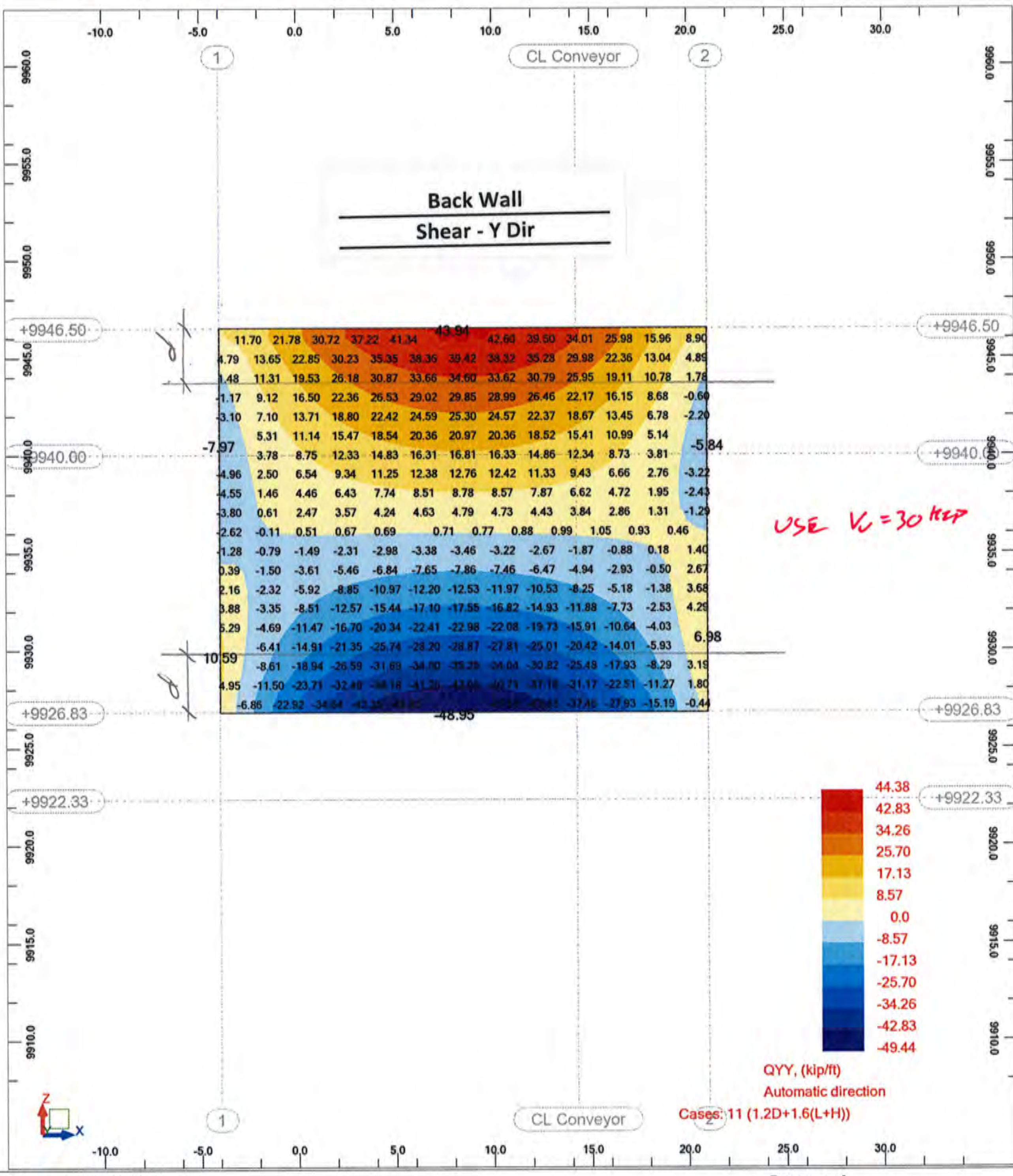
Project: **Vault - Rev C**
Client: **CC&V**

View - QYY (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))



USE $V_u = 30$ KIP

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Front Wall Reinforcement Requirement

per ACI 318-08

Material Properties:

t =	36	in	Total thickness of concrete
f'_c =	4	ksi	Compressive strength of concrete
λ =	1		Lightweight concrete factor
F_y =	60	ksi	Yield stress of reinforcement
ϕ_f =	0.9		Flexural reduction factor
ϕ_v =	0.75		Shear reduction factor

Flexural Requirement:

	X Direction		Y Direction			
	Outside	Inside	Outside*	Inside		
Mu =	58	25	90	50	kip*ft	Factored maximum moment
c =	2	2	2	2	in	Reinforcement cover
Bar:	# 8	# 8	# 8	# 8		Reinforcement provided
b =	12	12	12	12	in	Bar spacing
A_s =	0.79	0.79	0.79	0.79	in^2/ft	Provided steel reinforcement
d =	34	34	34	34	in	Distance to reinforcement
M_n =	132.01	132.01	132.01	132.01	k-ft	Nominal moment capacity
ϕM_n =	118.80	118.80	118.80	118.80	k-ft	Factored moment capacity
Ratio =	**0.49**	**0.21**	**0.76**	**0.42**		

Shear Requirement

	X Dir.	Y Dir.		
V_u =	15	20	kip/ft	Factored maximum shear
V_n =	51.6	51.6	kip/ft	Nominal shear capacity per ft widtl
ϕV_n =	38.71	38.71	kip/ft	Factored shear capacity per ft wid
Ratio =	**0.39**	**0.52**		

Temperature Requirement

ρ =	0.0018	0.0018		Required percentage for temperature steel
ρ_w =	0.0037	0.0037	%	Reinforcement provided
$A_{s,Required}$ =	0.7776	0.7776	in^2	
	OK	**OK**		

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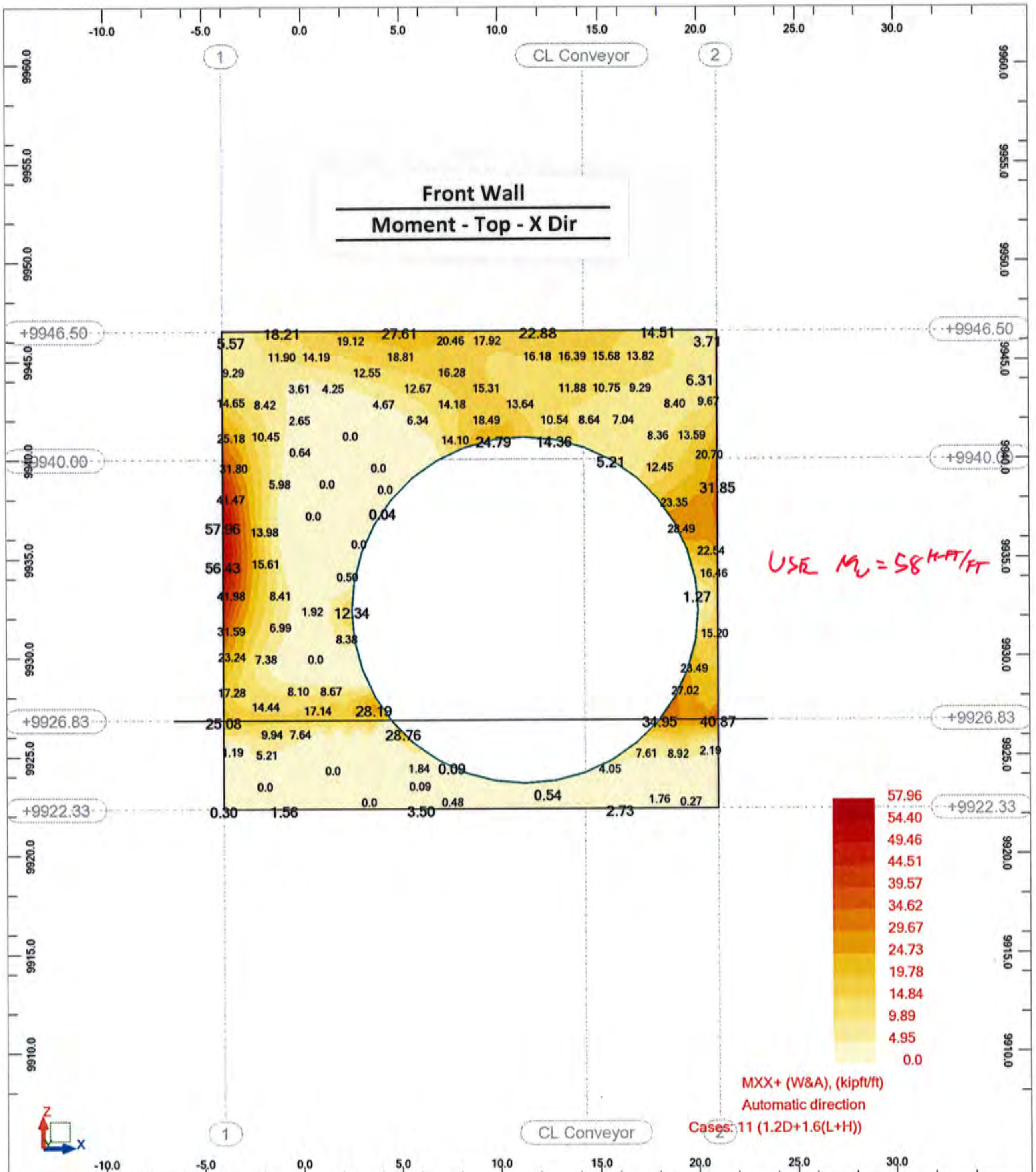
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX+ (W&A) Cases: 11 (1.2D+1.6(L+H))



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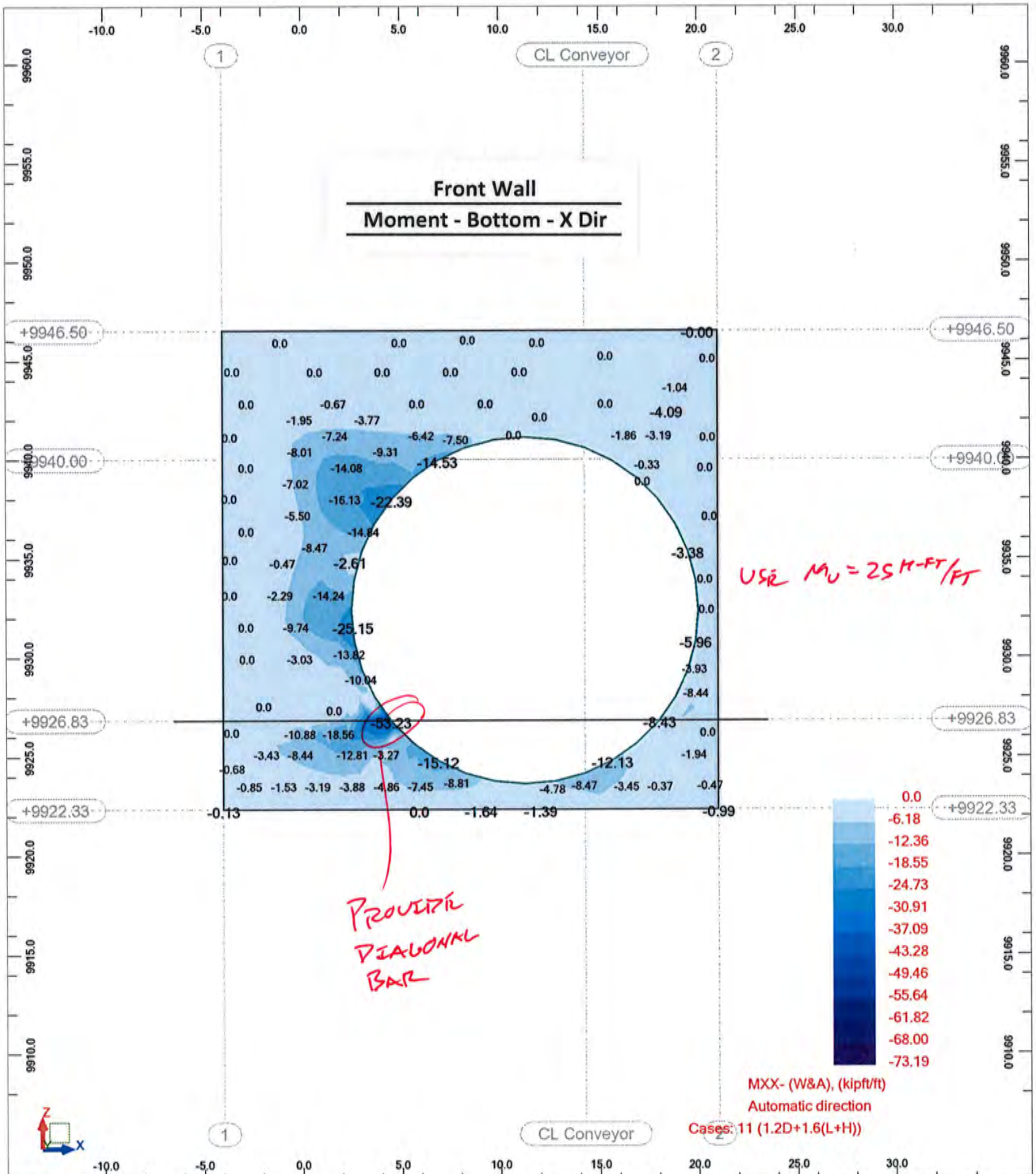
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX- (W&A) Cases: 11 (1.2D+1.6(L+H))



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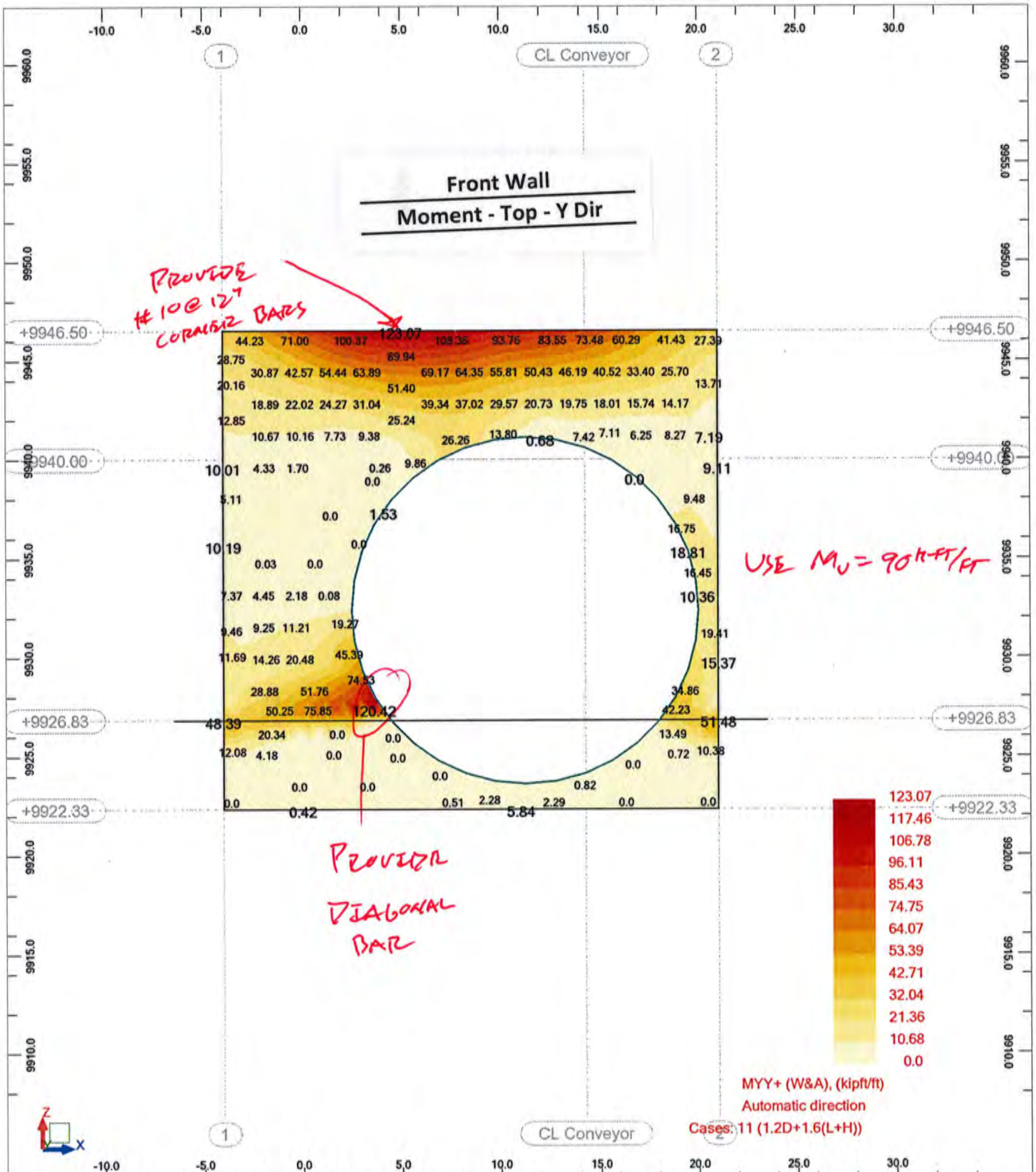
Project: **Vault - Rev C**
Client: **CC&V**

View - MYY+ (W&A) Cases: 11 (1.2D+1.6(L+H))



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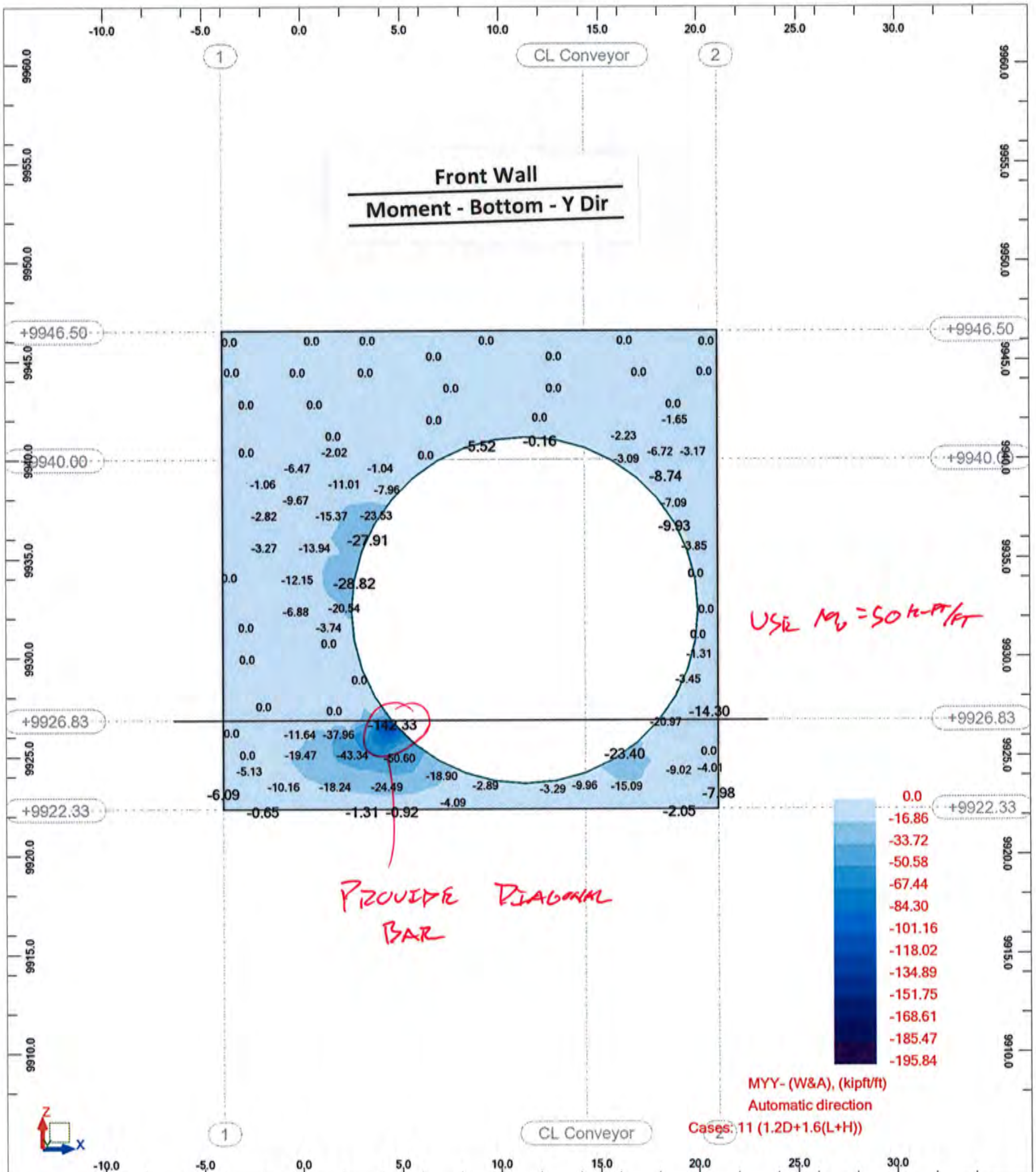
Client: **CC&V**

View - MYY- (W&A) Cases: 11 (1.2D+1.6(L+H))



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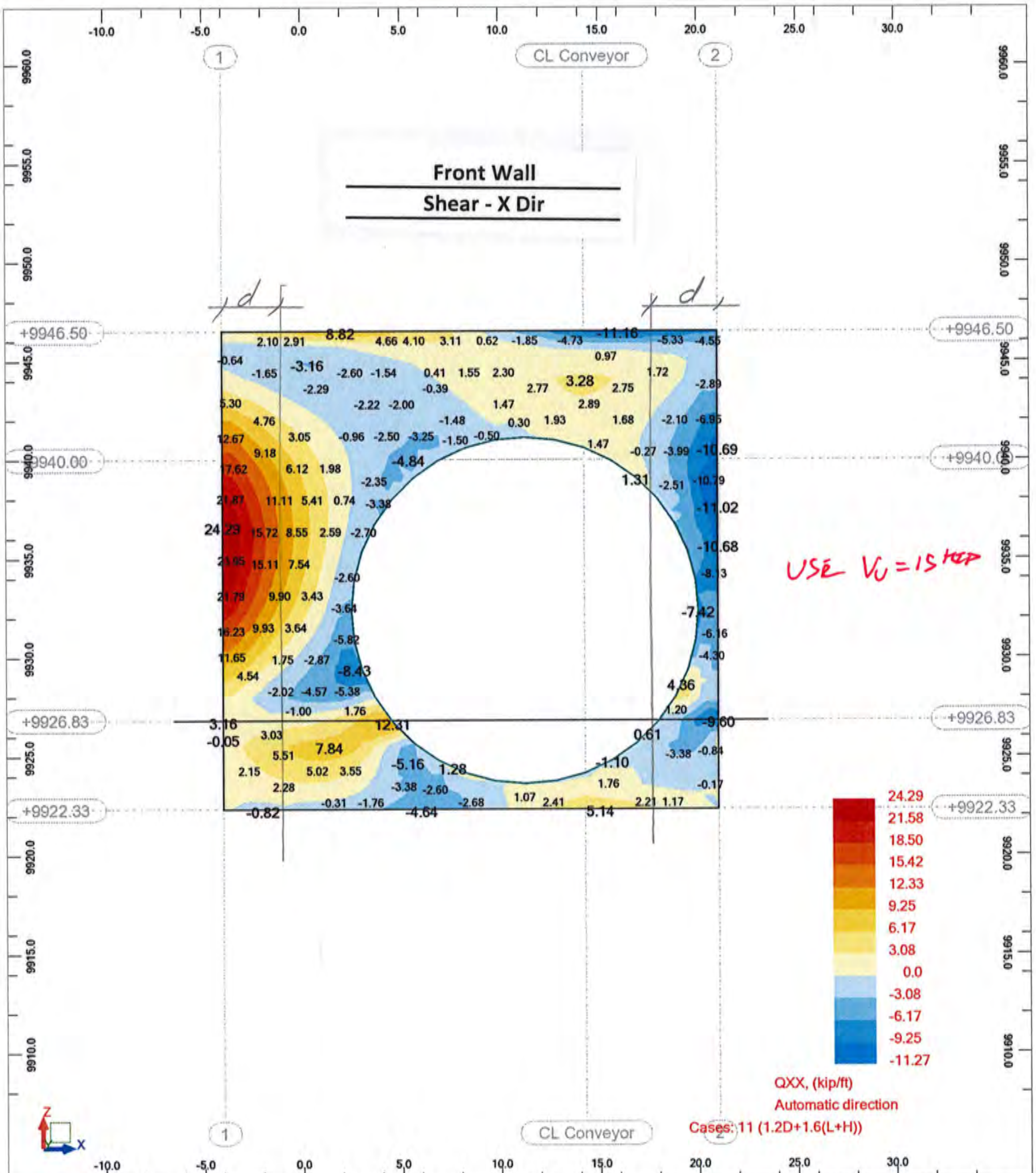
Client: **CC&V**

View - QXX (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))



USE $V_U = 15$ kip

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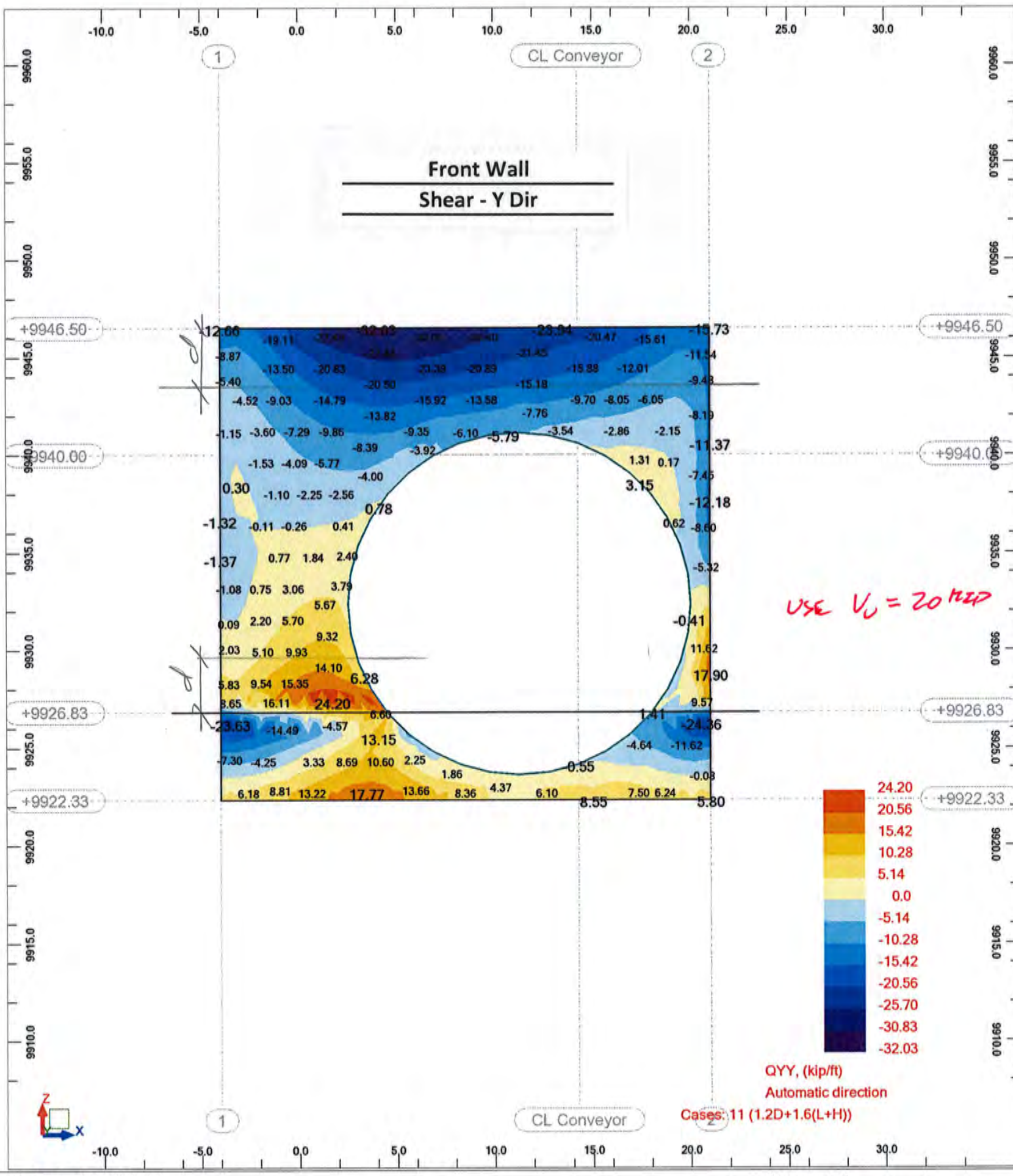
Project: **Vault - Rev C**
Client: **CC&V**

View - QYY (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))



USE $V_u = 20$ KIP

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Side (North) Wall Reinforcement Requirement

per ACI 318-08

Material Properties:

$t =$	36	in	Total thickness of concrete
$f'_c =$	4	ksi	Compressive strength of concrete
$\lambda =$	1		Lightweight concrete factor
$F_y =$	60	ksi	Yield stress of reinforcement
$\phi_f =$	0.9		Flexural reduction factor
$\phi_v =$	0.75		Shear reduction factor

Flexural Requirement:

	X Direction		Y Direction			
	Outside	**Inside**	**Outside***	**Inside**		
Mu =	64	25	150	40	kip*ft	Factored maximum moment
c =	2	2	2	2	in	Reinforcement cover
Bar:	# 8	# 8	# 10	# 8		Reinforcement provided
b =	12	12	12	12	in	Bar spacing
$A_s =$	0.79	0.79	1.27	0.79	in^2/ft	Provided steel reinforcement
d =	34	34	34	34	in	Distance to reinforcement
$M_n =$	132.01	132.01	209.97	132.01	k-ft	Nominal moment capacity
$\phi M_n =$	118.80	118.80	188.97	118.80	k-ft	Factored moment capacity
Ratio =	**0.54**	**0.21**	**0.79**	**0.34**		

Shear Requirement

	X Dir.	Y Dir.		
$V_u =$	15	35	kip/ft	Factored maximum shear
$V_n =$	51.6	51.6	kip/ft	Nominal shear capacity per ft widtl
$\phi V_n =$	38.71	38.71	kip/ft	Factored shear capacity per ft widt
Ratio =	**0.39**	**0.90**		

Temperature Requirement

$\rho =$	0.0018	0.0018		Required percentage for temperature steel
$\rho_w =$	0.0037	0.0048	%	Reinforcement provided
$A_{s,Required} =$	0.7776	0.7776	in^2	
	OK	**OK**		

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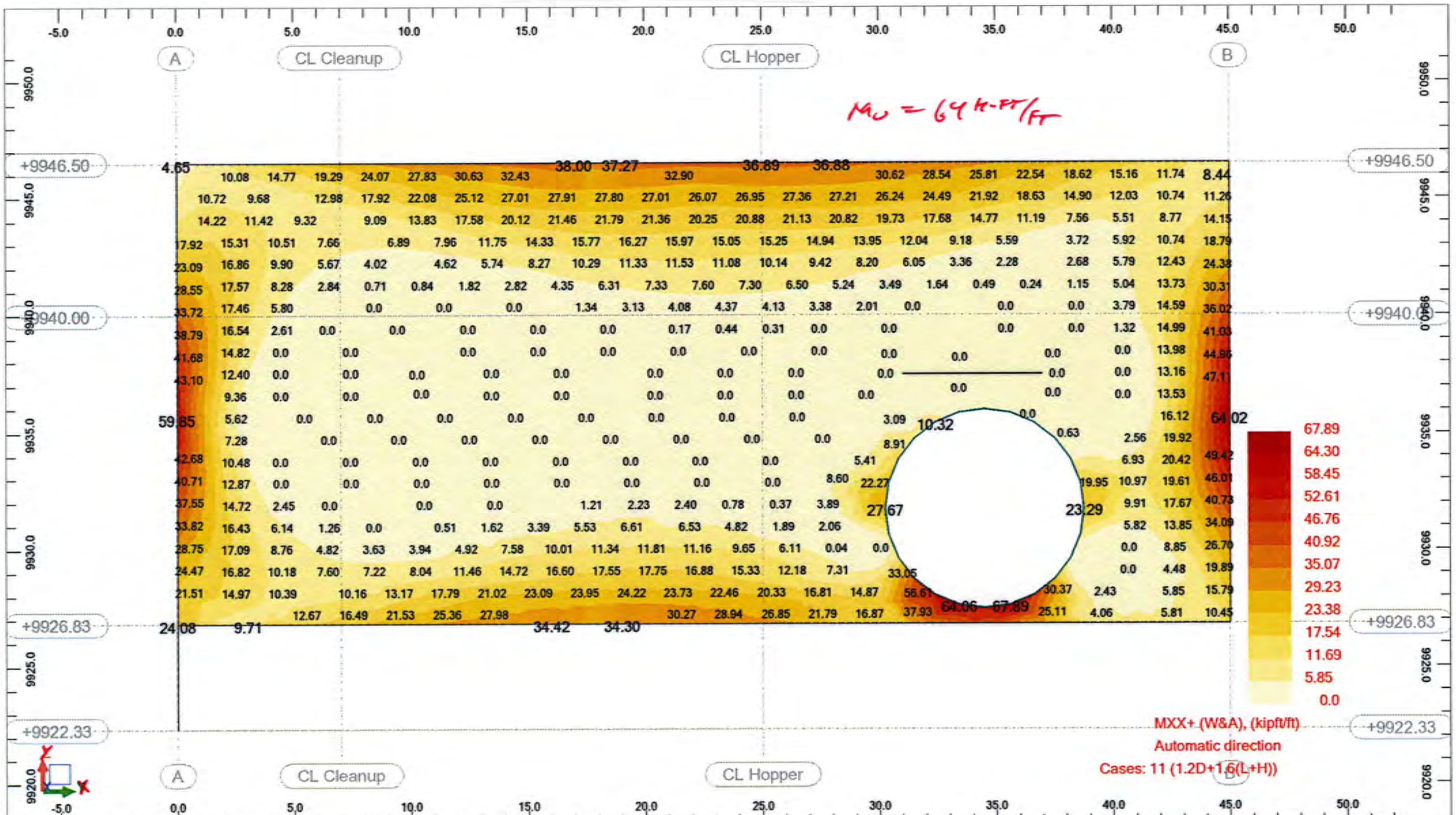
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (North)

Moment - Top - X Dir



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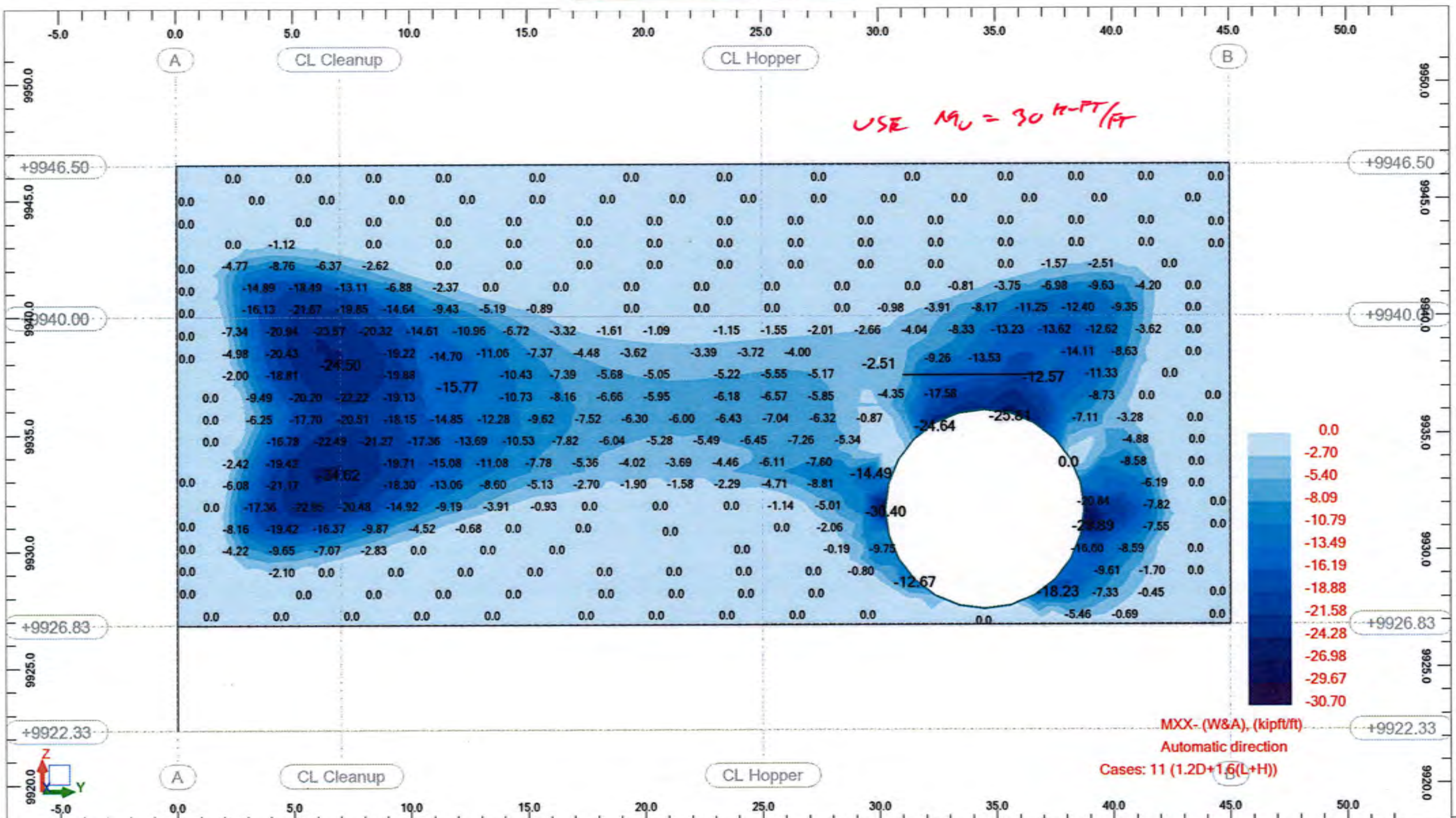
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX- (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (North)

Moment - Bottom - X Dir



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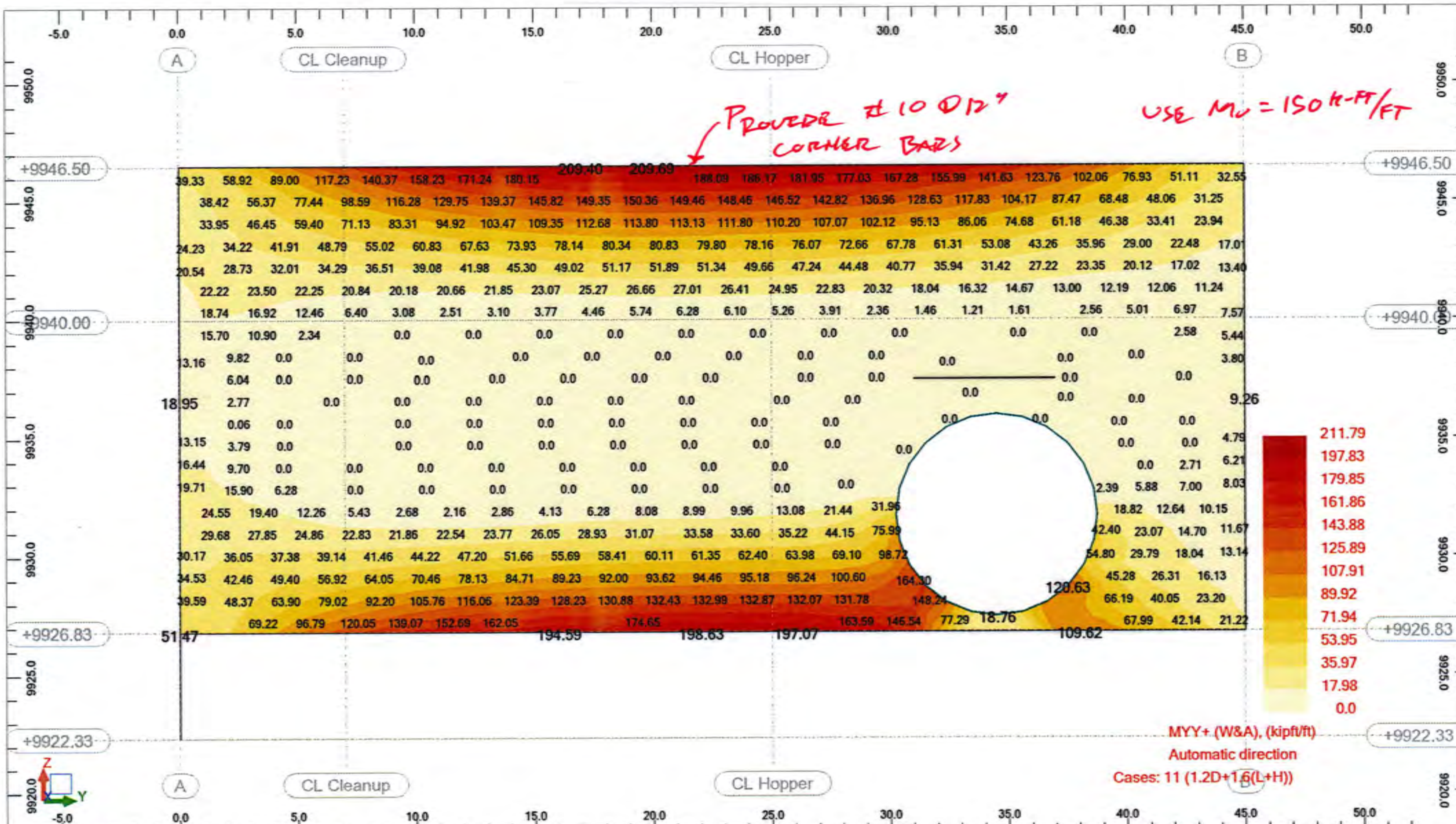
Client: **CC&V**

View - MYY+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (North)

Moment - Top - Y Dir



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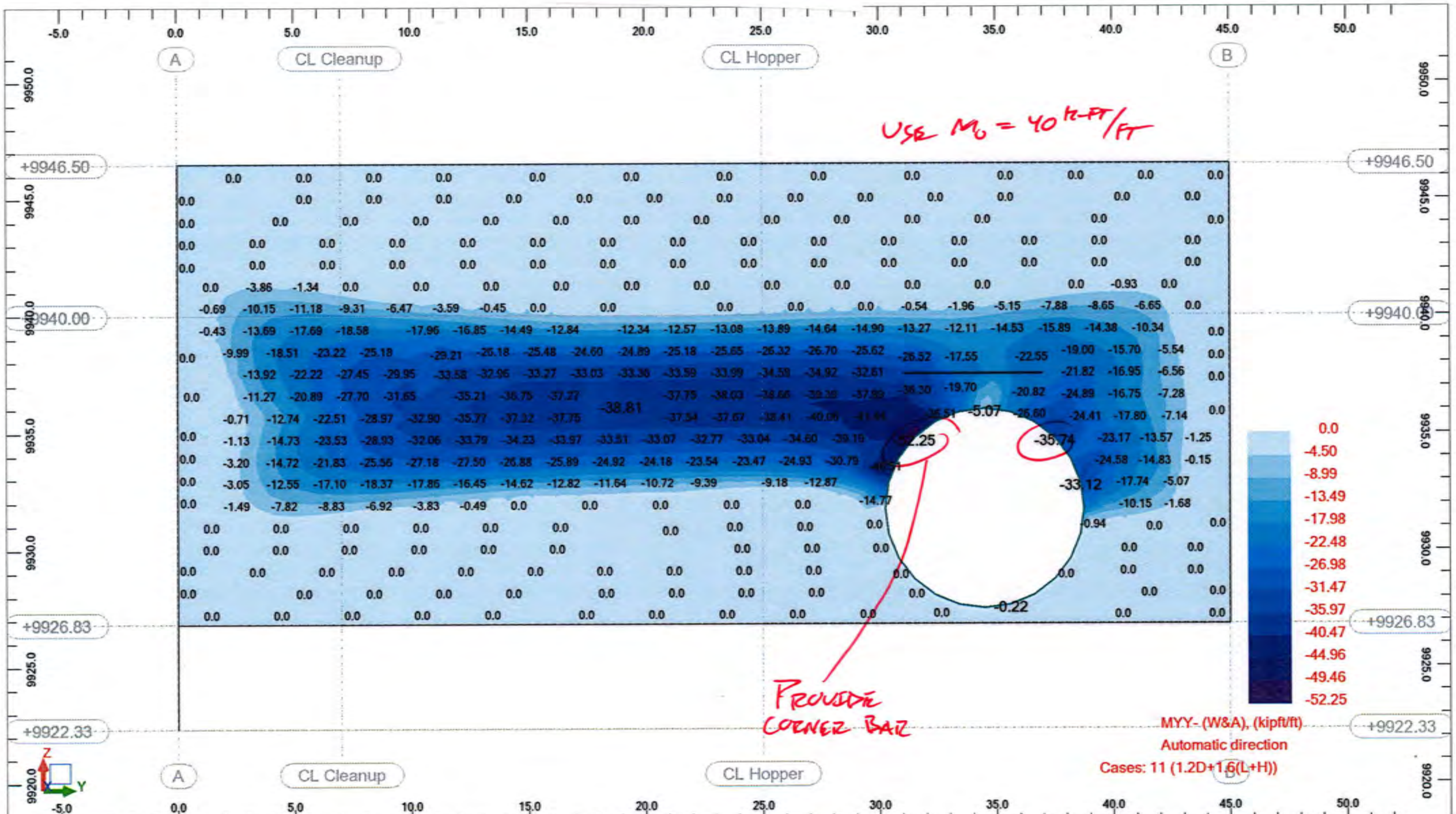
Client: **CC&V**

View - MYY- (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (North)

Moment - Bottom - Y Dir



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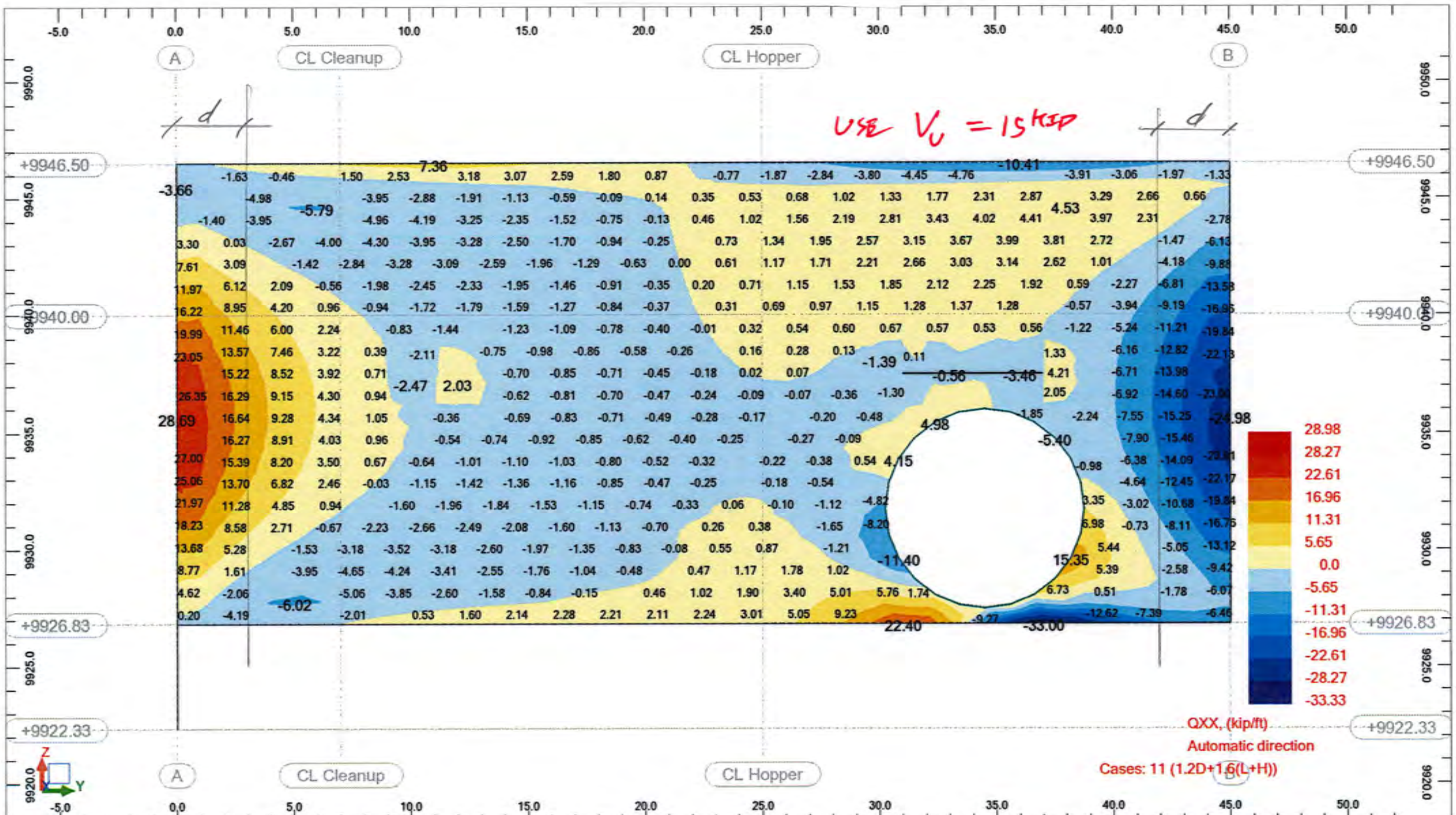
Client: **CC&V**

View - QXX (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Side Wall (North)

Shear - X Dir



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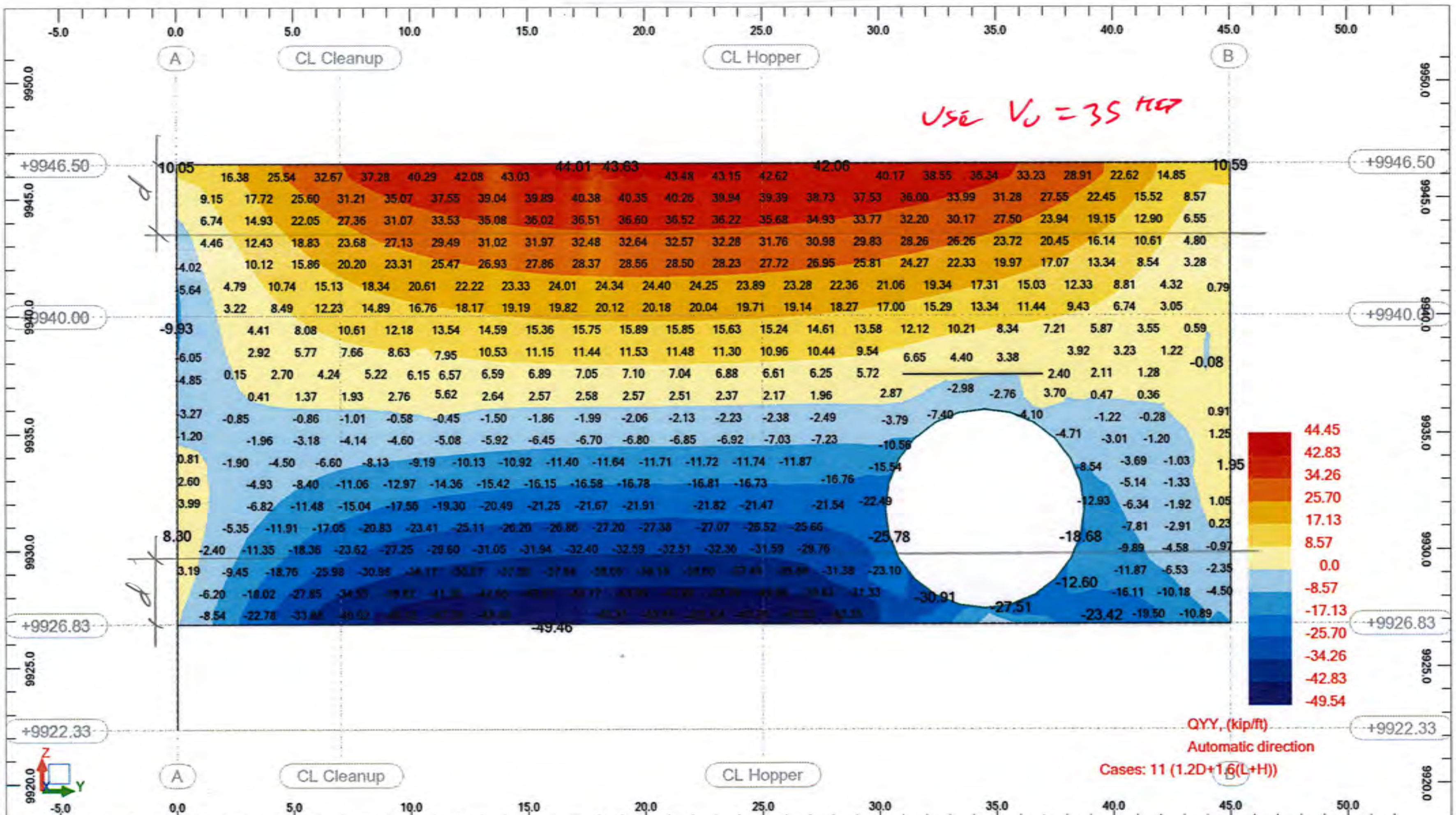
Client: **CC&V**

View - QYY (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Side Wall (North)

Shear - Y Dir



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Side (South) Wall Reinforcement Requirement

per ACI 318-08

Material Properties:

$t =$	36	in	Total thickness of concrete
$f'_c =$	4	ksi	Compressive strength of concrete
$\lambda =$	1		Lightweight concrete factor
$F_y =$	60	ksi	Yield stress of reinforcement
$\phi_f =$	0.9		Flexural reduction factor
$\phi_v =$	0.75		Shear reduction factor

Flexural Requirement:

	X Direction		Y Direction			
	Outside	Inside	Outside*	Inside		
Mu =	69	28	160	40	kip*ft	Factored maximum moment
c =	2	2	2	2	in	Reinforcement cover
Bar:	# 8	# 8	# 10	# 8		Reinforcement provided
b =	12	12	12	12	in	Bar spacing
$A_s =$	0.79	0.79	1.27	0.79	in^2/ft	Provided steel reinforcement
d =	34	34	34	34	in	Distance to reinforcement
$M_n =$	132.01	132.01	209.97	132.01	k-ft	Nominal moment capacity
$\phi M_n =$	118.80	118.80	188.97	118.80	k-ft	Factored moment capacity
Ratio =	**0.58**	**0.24**	**0.85**	**0.34**		

Shear Requirement

	X Dir.	Y Dir.		
$V_u =$	15	35	kip/ft	Factored maximum shear
$V_n =$	51.6	51.6	kip/ft	Nominal shear capacity per ft widtl
$\phi V_n =$	38.71	38.71	kip/ft	Factored shear capacity per ft widt
Ratio =	**0.39**	**0.90**		

Temperature Requirement

$\rho =$	0.0018	0.0018		Required percentage for temperature steel
$\rho_w =$	0.0037	0.0048	%	Reinforcement provided
$A_{s,Required} =$	0.7776	0.7776	in^2	
	OK	**OK**		

FLSMIDTH

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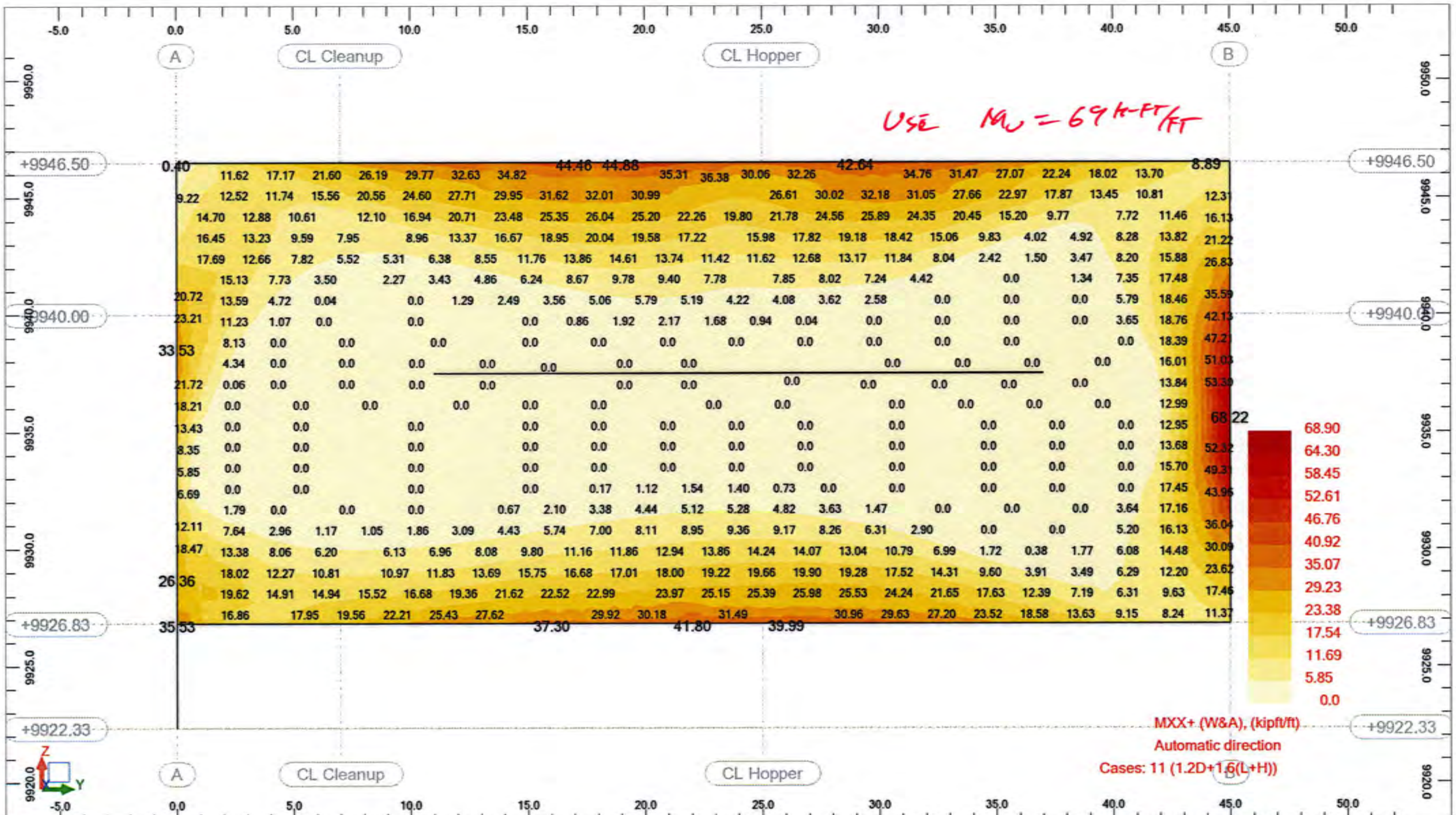
View - MXX+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

Side Wall (South)

Moment - Top - X Dir



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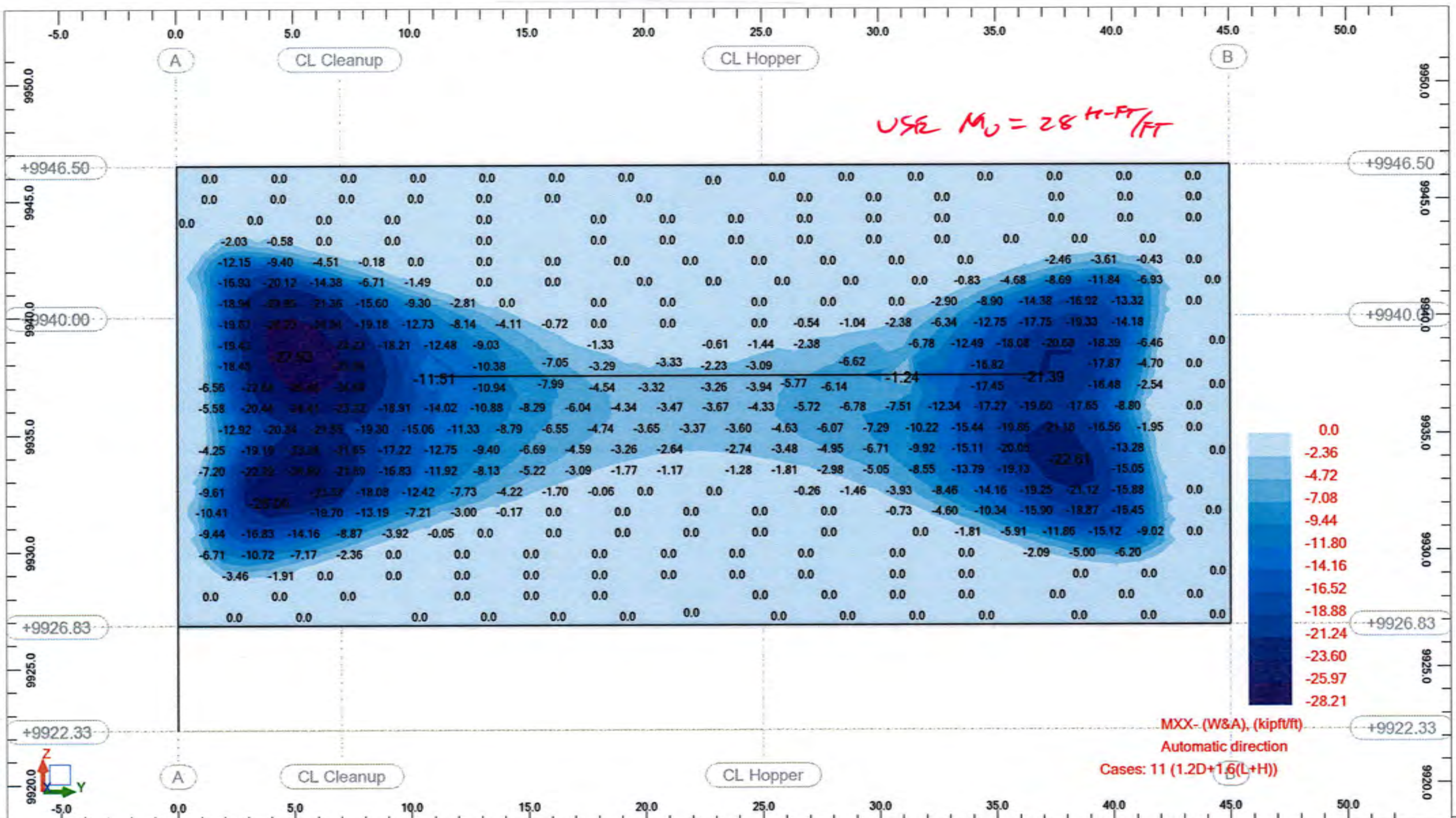
Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - MXX- (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (South)

Moment - Bottom - X Dir



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Project: **Vault - Rev C**
Client: **CC&V**

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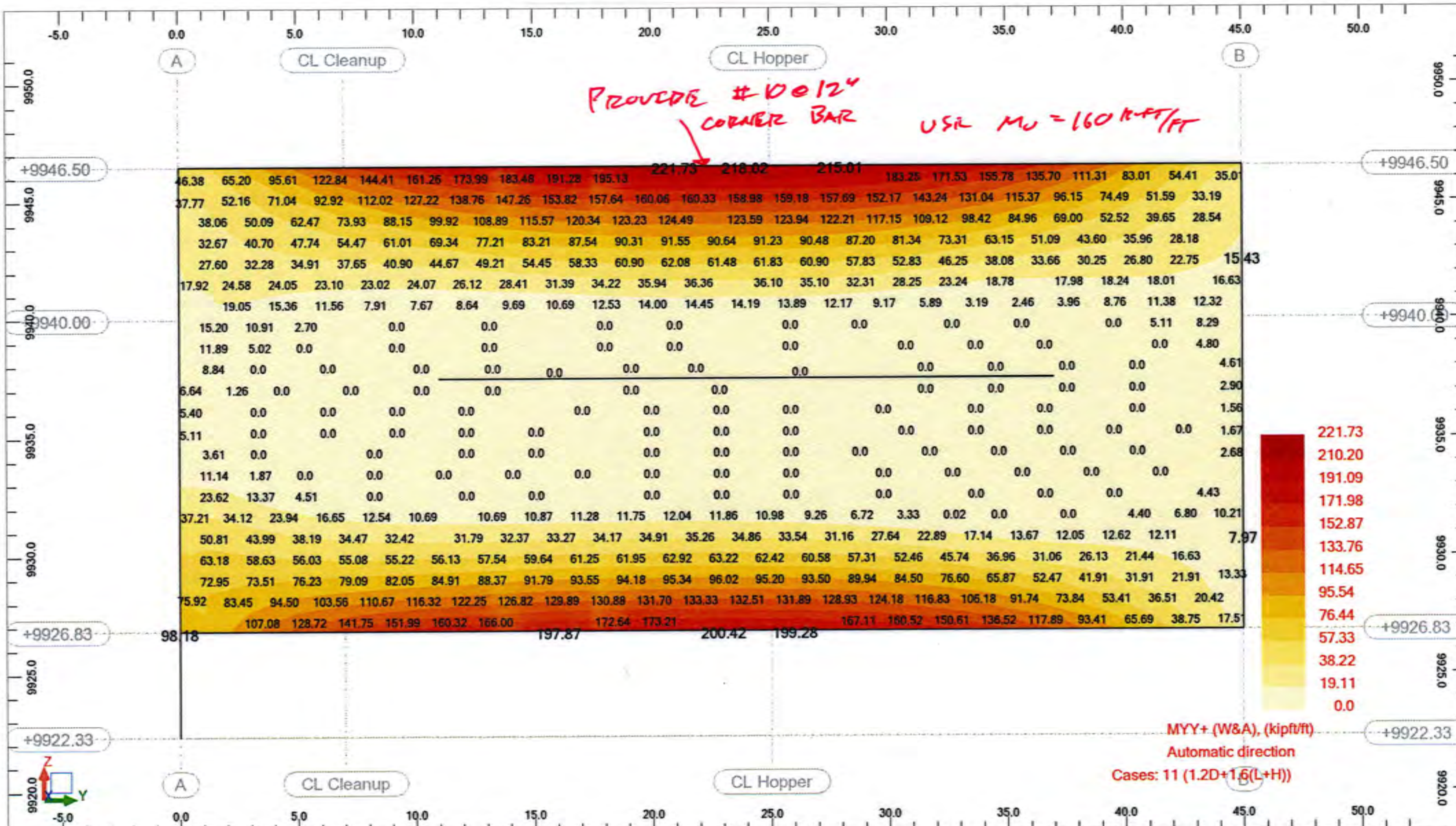
View - MYY+ (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (South)

Moment - Top - Y Dir



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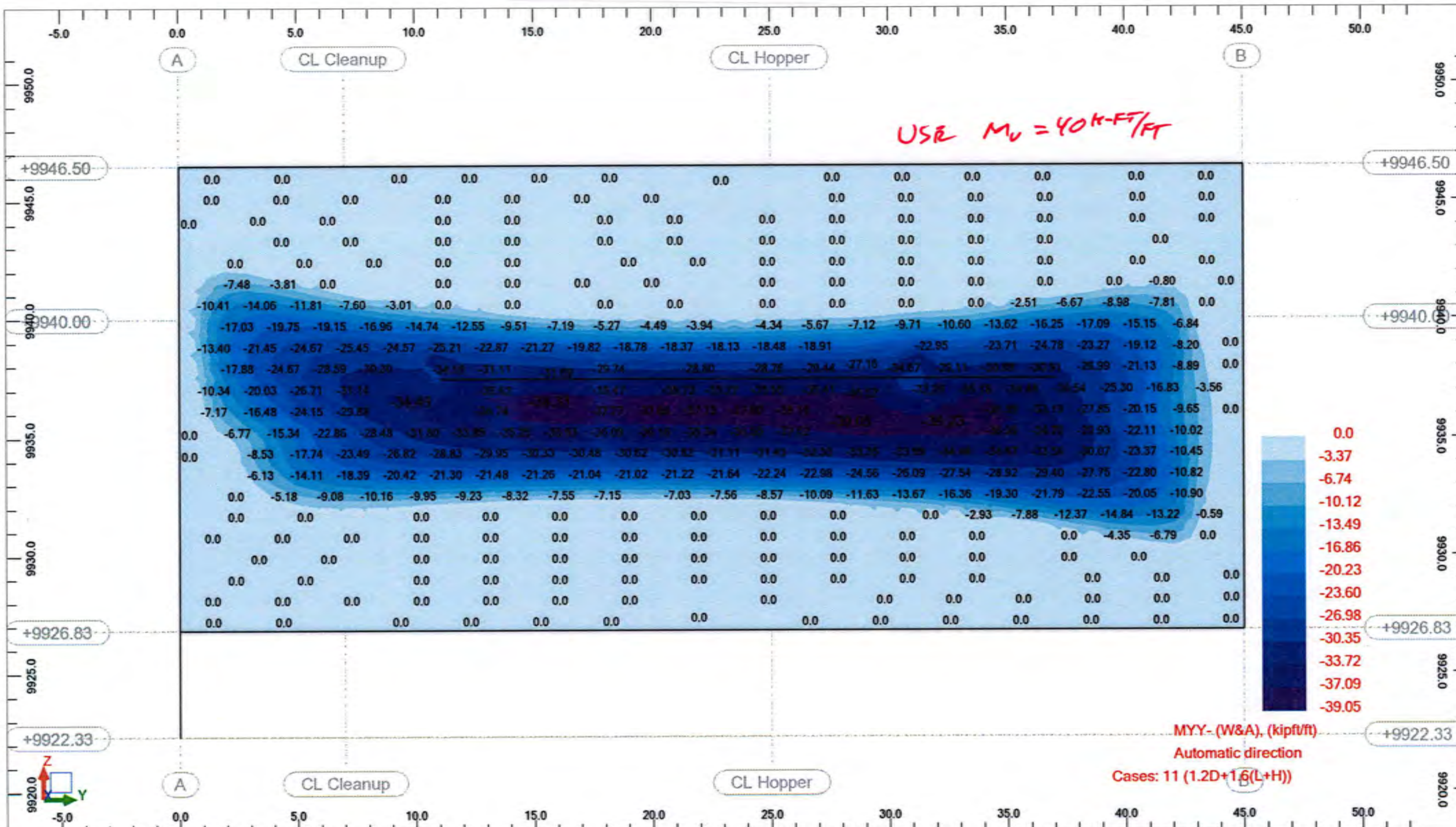
Client: **CC&V**

View - MYY- (W&A) Cases: 11 (1.2D+1.6(L+H))

Side Wall (South)

Moment - Bottom - Y Dir



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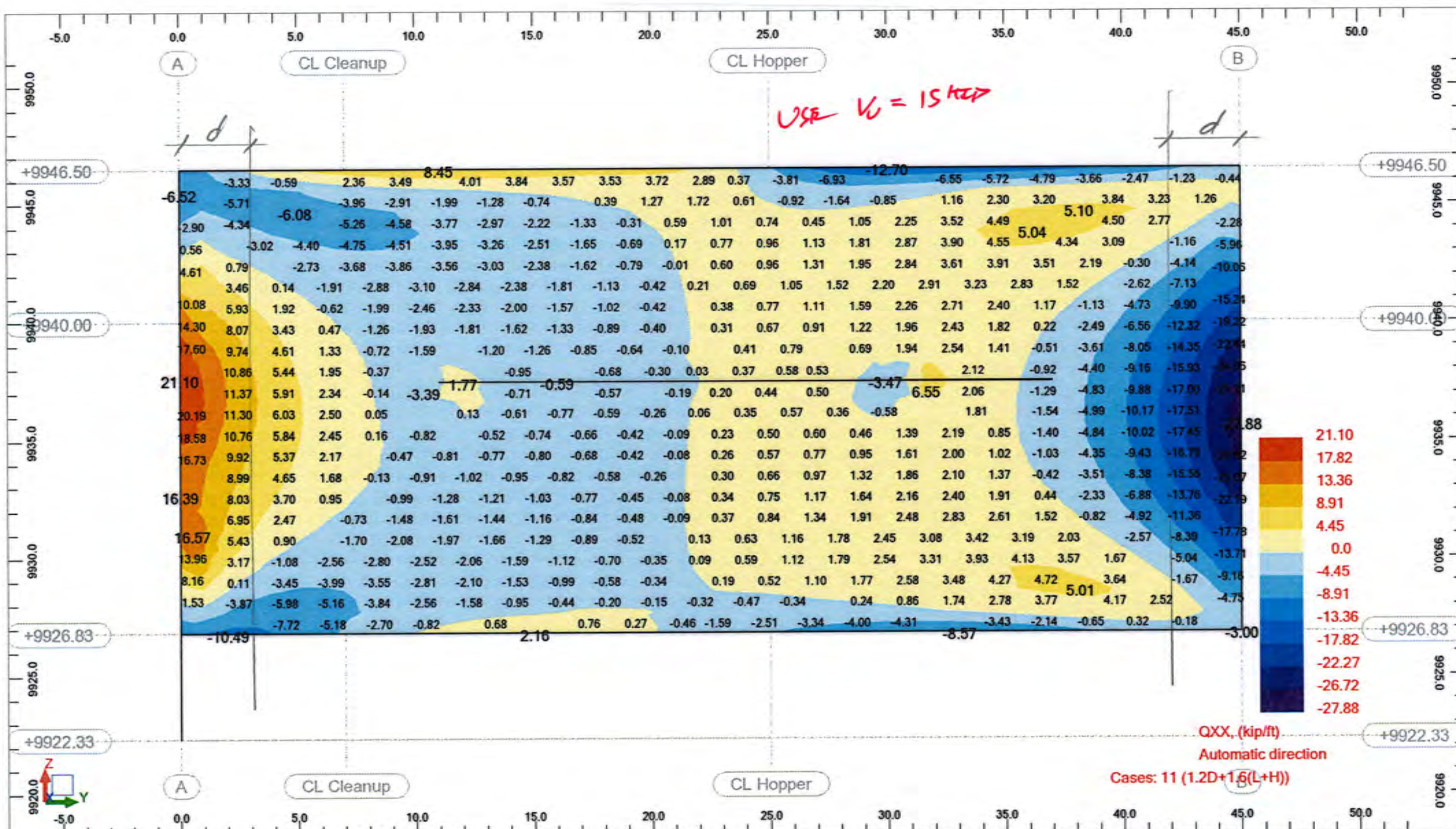
Project: **Vault - Rev C**
Client: **CC&V**

View - QXX (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Side Wall (South)

Shear - X Dir



Date : 30/09/11
File: **Vault - Rev C.rtd**

Page : **1**
Robot 2012 © AutoDesk

FLSmidth

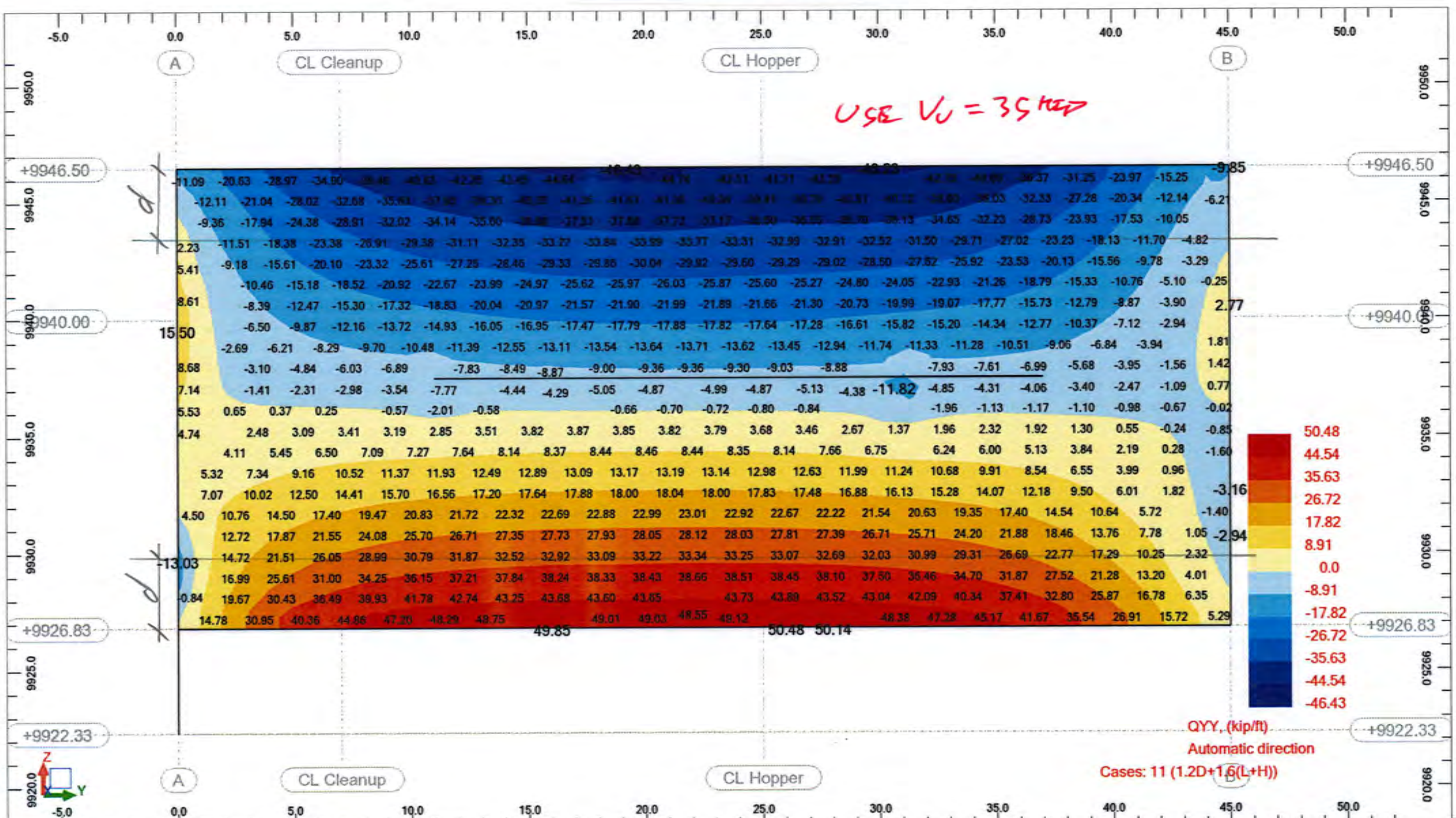
7158 S. FLSmidth Drive
Midvale, UT 84047-5559

Engineer: **Ed Summmers**
Calc No..: **C-002** Job.:**11021**
Project: **Vault - Rev C**
Client: **CC&V**

View - QYY (kip/ft) Automatic direction Cases: 11 (1.2D+1.6(L+H))

Side Wall (South)

Shear - Y Dir

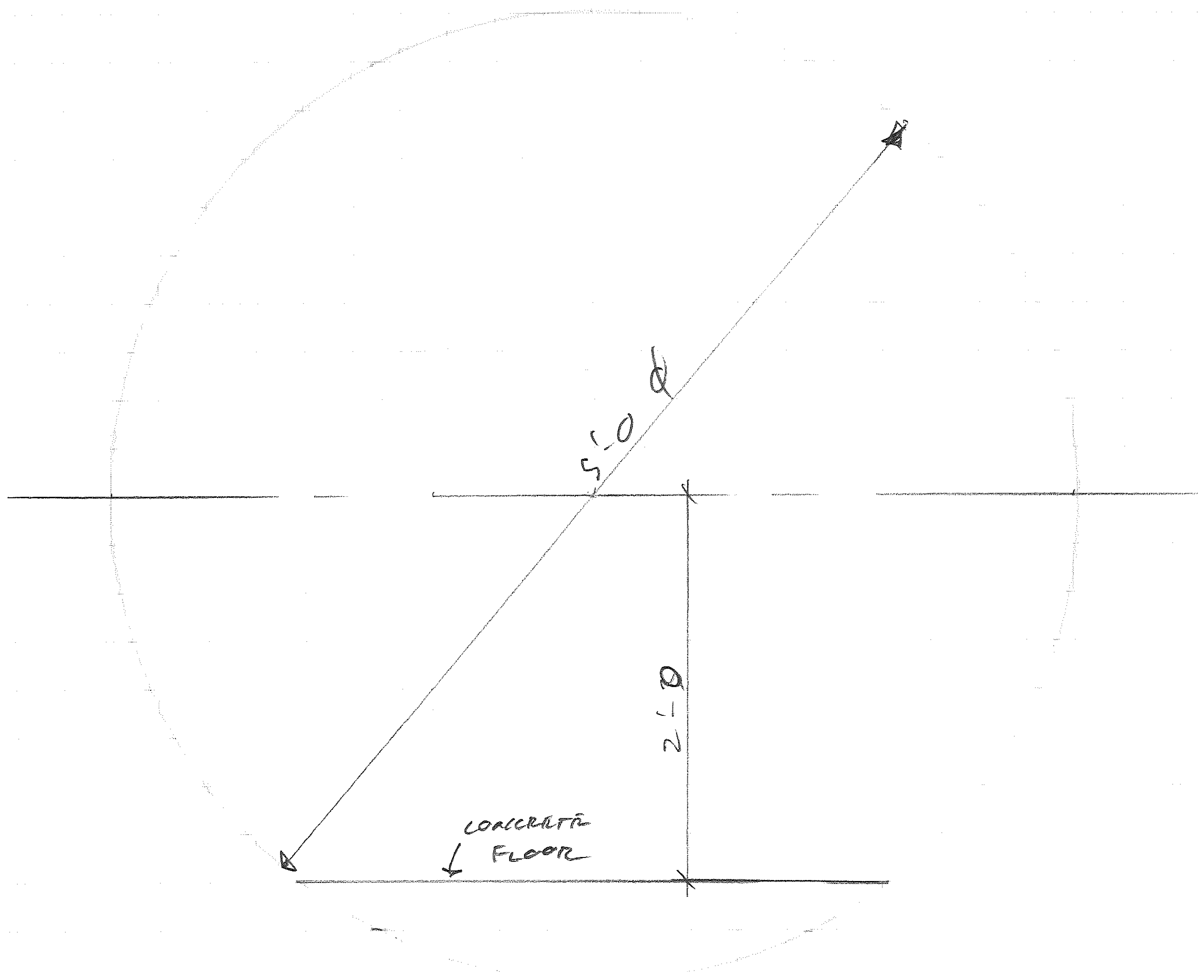


Date : 30/09/11
File: **Vault - Rev C.rtd**

Page : **1**
Robot 2012 © AutoDesk

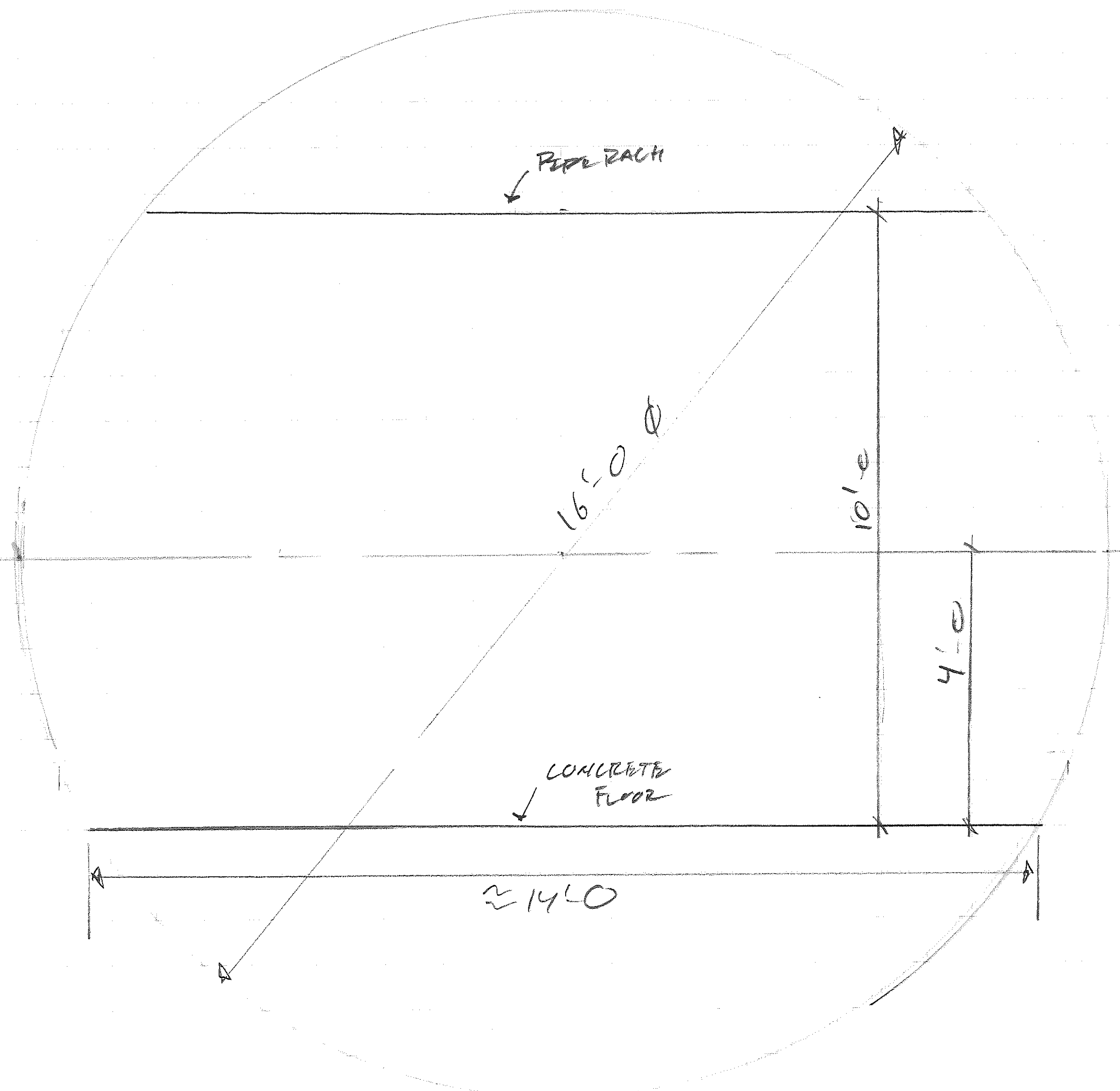
FLSMIDTH	PROJECT CC & V HIGH GRADE MILL	PROJ NO. 11021	
		CALC NO. C-002	
	SUBJECT RECLAIM VAULT TUNNELS	BY RCS	DATE 9-28-11
		CHK	DATE
		SHEET OF	REV

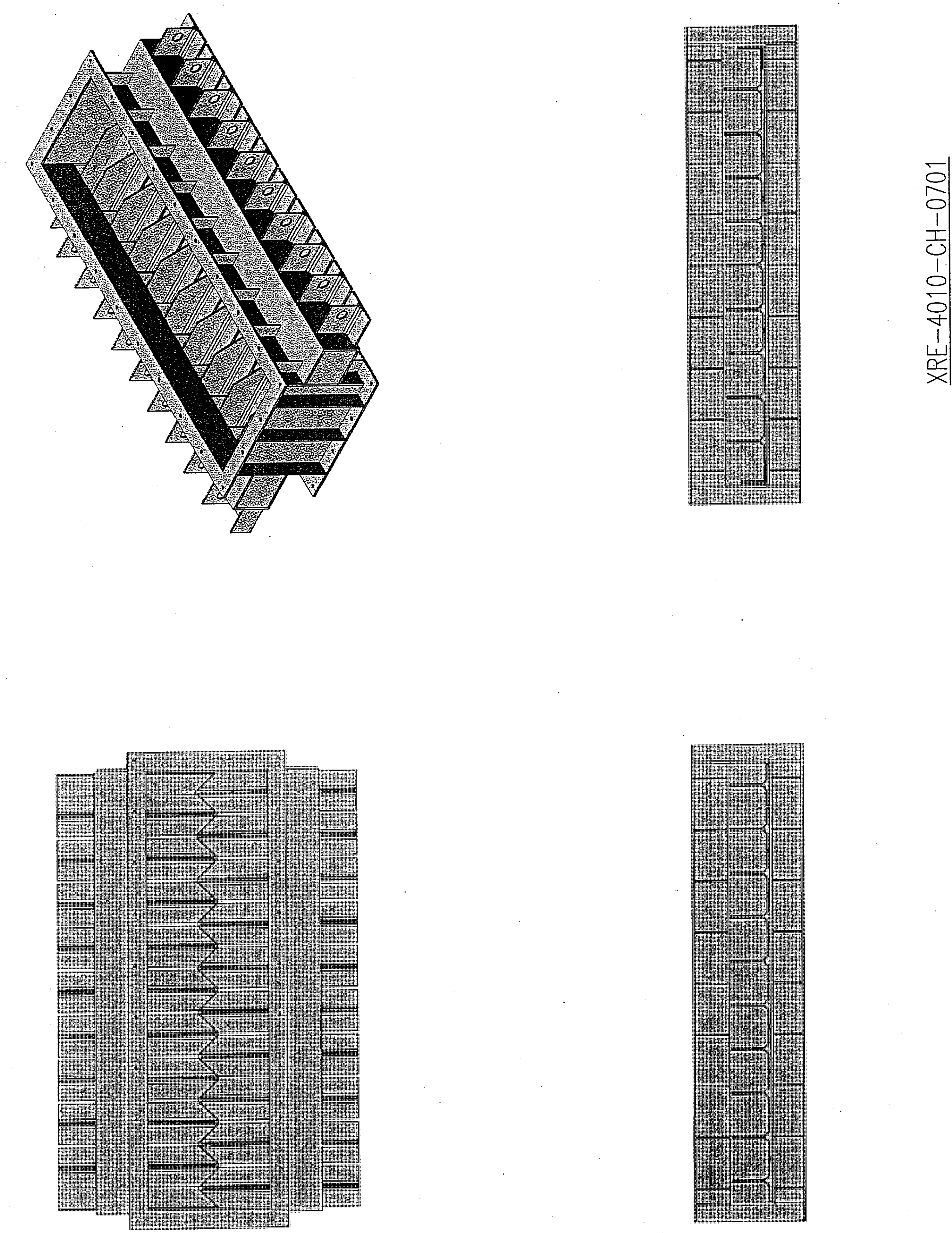
EGRESS TUNNEL

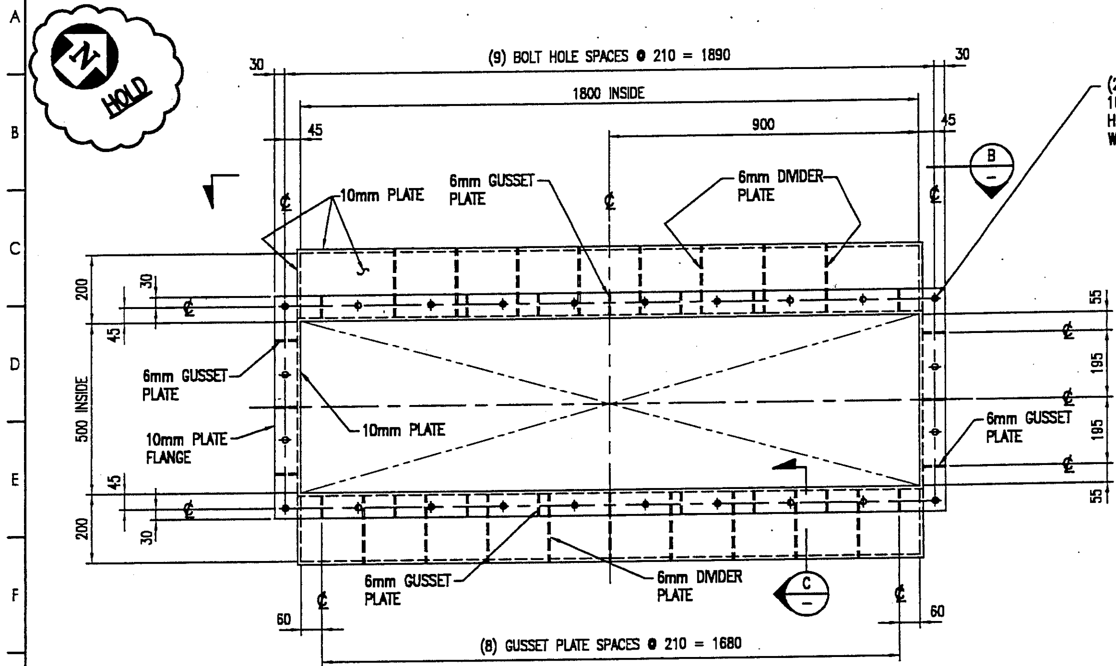


PROJECT CC&V HIGH GRADE ORE	PROJ NO. 11021
	CALC NO. C-002
SUBJECT RECLAIM AREA TUNNEL	BY RCS DATE 9-28-11
	CHK DATE
	SHEET OF REV

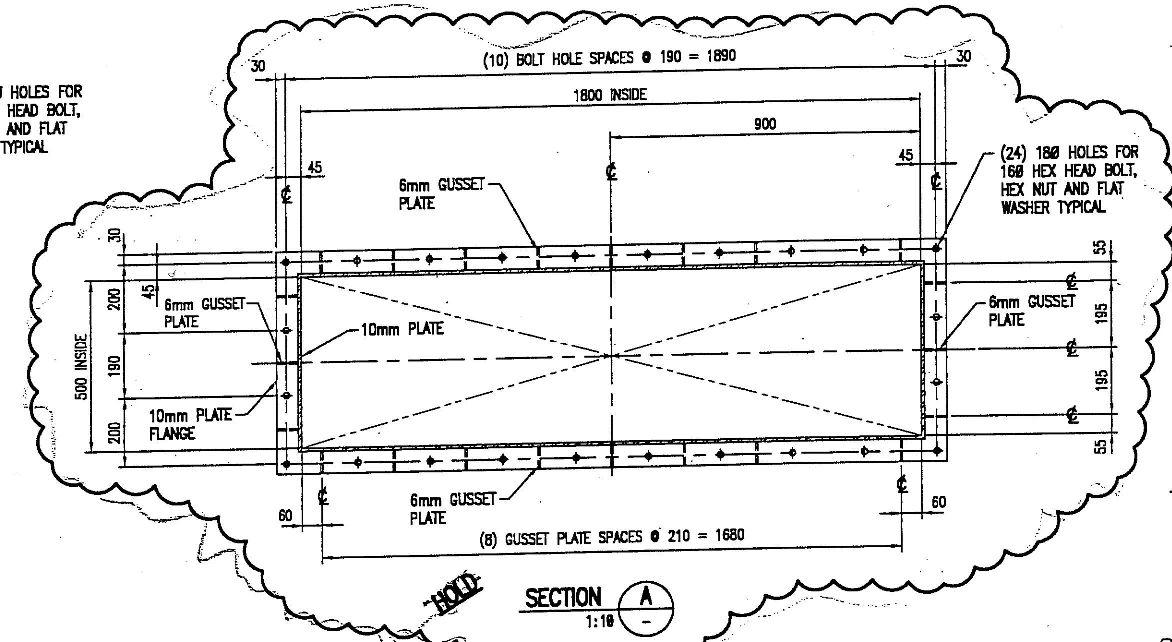
RECLAIM TUNNEL



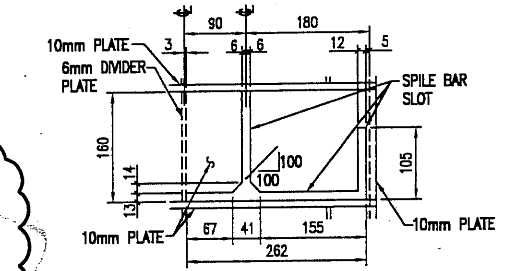

XRE-4010-CH-0701



PLAN

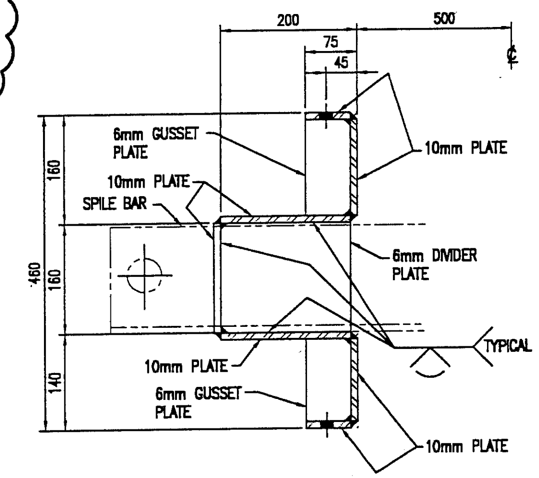


SECTION A 1:10

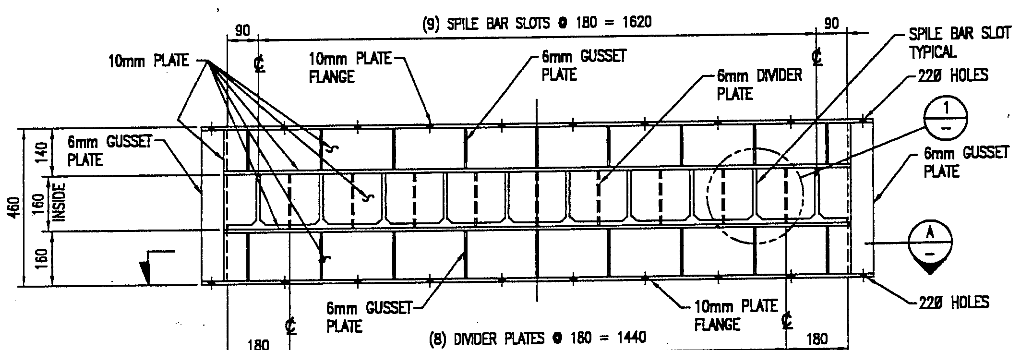


DETAIL 2 1:5 SPILE BAR SLOT (1) REQUIRED AS SHOWN

DETAIL 3 1:5 SPILE BAR SLOT (1) REQUIRED OPPOSITE HAND

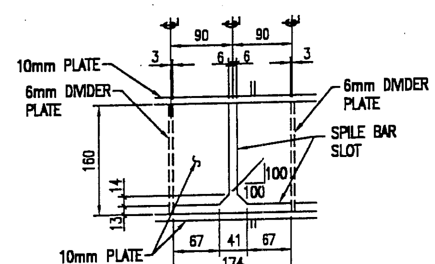


SECTION C 1:5

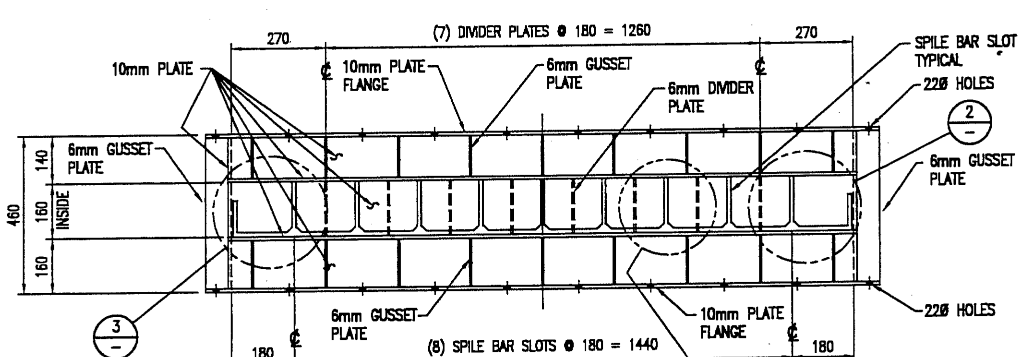


ELEVATION

SPILE BAR CHUTE 4010-CH-0701 DETAIL
SCALE 1:10 (SPILE BARS NOT SHOWN FOR CLARITY)



DETAIL 1 1:5 SPILE BAR SLOT (17) REQUIRED.



VIEW B 1:10

NOTES:

1. CHUTE TO BE ALL WELDED CONSTRUCTION. ALL WELDS TO BE CONTINUOUS FILLET WELDS UNLESS NOTED OTHERWISE.
2. ALL DIMENSIONS ARE TO THE INSIDE UNLESS NOTED OTHERWISE.
3. CHUTE WALLS TO BE 10 THK. ASTM A36 STEEL.
4. FOR SPILE BAR ARRANGEMENT AND DETAILS SEE DWG. DT-10082-M-115.

SOUMIS POUR : ☐ INFORMATION (F) ☐ REVUE (R)

LA REVUE NE CONSTITUE PAS L'ACCEPTATION DES DÉTAILS DE LA CONCEPTION, DES CALCULS, DES MÉTHODES D'ESSAI NI DES MATÉRIAUX MIS AU POINT OU CHOISIS PAR LE FOURNISSEUR. ELLE NE LIBÈRE PAS NON PLUS LE FOURNISSEUR DE SON OBLIGATION DE SE CONFORMER INTÉGRALEMENT À SES OBLIGATIONS PRÉVUES AU CONTRAT NI À AUCUNE AUTRE DE SES OBLIGATIONS.

- ☐ 1. REVUE, SANS COMMENTAIRES, NE PAS RE SOUMETTRE.
- ☐ 2. REVUE, SANS COMMENTAIRES, RE SOUMETTRE COPIE CERTIFIÉE TEL LE LDRF ("VDRL").
- ☐ 3. REVUE, POURSUIVRE LE TRAVAIL, INSÉRER LES COMMENTAIRES EN T, RE SOUMETTRE.
- ☐ 4. NON ACCEPTÉ. TRAVAIL INTERROMPU. REVISER ET, RE SOUMETTRE.
- ☐ 5. POUR INFORMATION SEULEMENT.

NOM : ____________ DATE : ____________

SIGNATURE : ____________

NOM DU PROJET : USINE D'ENRICHISSEMENT GUELB II

NOM DU FOURNISSEUR : FLSMIDTH

DESCRIPTION DU LOT : WET CONCENTRATE BUILDING

Nos D'ÉTIQUETTE :

Nos DE MODÈLE : 4010-CH-0701

CLASSIFICATION :

N° DE PROJET	N° DE BON DE COMMANDE	CODE LDRF	N° SÉQUENTIEL	N° DE SOUMISSION
020486	P-40-5004	D06	0005	01

N° DE DOCUMENT SLI

SNC-LAVALIN
SNC•LAVALIN 455, Boul. René-Lévesque Ouest Montréal (Québec) Canada H2Z 1Z3

CLIENT Snim el Rhein II MAURITANIE

ISSUE RECORD

DATE	ISS	REV	TO	FOR

1	—	—	—	INITIAL RELEASE

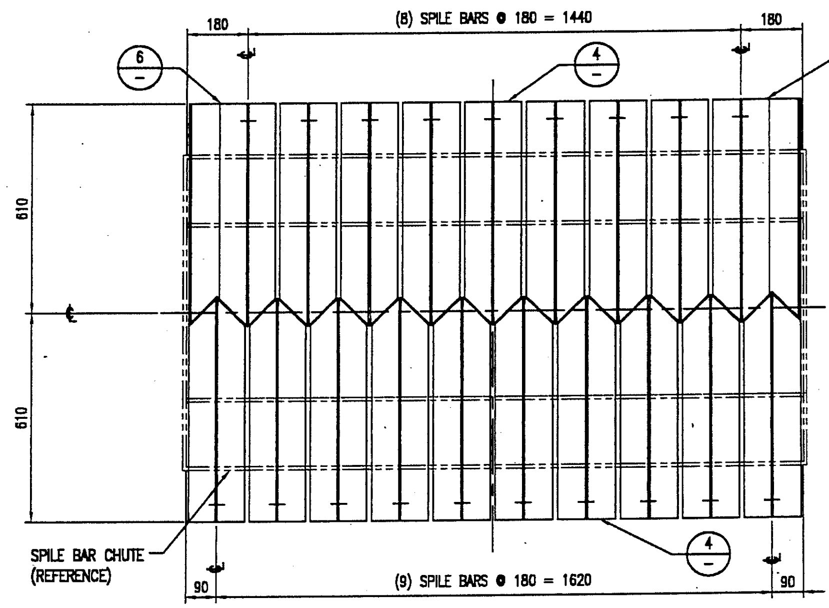
Scale	Drawn	Appr.	Appr. Date	Zone/Description	
NOTED	JEF	—	2/15/2011	Sheet Size: A1	Sheet 1 of 2

SNIM GUELB II BENEFICIATION
WET CONCENTRATE BUILDING
4010-CH-0701 CHUTE SPILE BARS
DETAILS

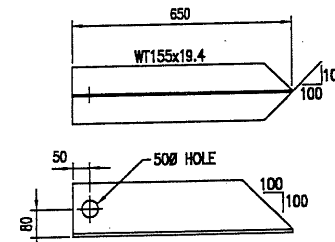
Mat.: NA Weight: NA Welds: NA

FLSMIDTH No.: DT-10082-M-114 Rev.: 1.01

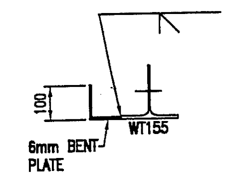
HOLD

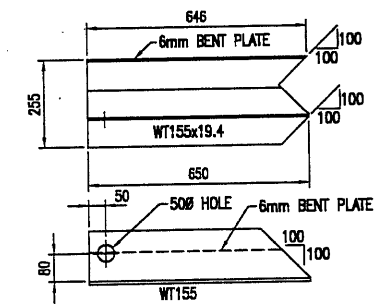


SPILE BAR ARRANGEMENT
SCALE 1:10



DETAIL 4 SPILE BARS (17) REQUIRED.
1:10





DETAIL 5 SPILE BAR (1) REQUIRED AS SHOWN
1:10

DETAIL 6 SPILE BAR (1) REQUIRED OPPOSITE HAND
1:10

NOTES:

1. CHUTE TO BE ALL WELDED CONSTRUCTION. ALL WELDS TO BE CONTINUOUS FILLET WELDS UNLESS NOTED OTHERWISE.
2. ALL DIMENSIONS ARE TO THE INSIDE UNLESS NOTED OTHERWISE.
3. CHUTE WALLS TO BE 10 THK. ASTM A36 STEEL.
4. FOR SPILE BAR CHUTE DETAILS SEE DWG. DT-10082-M-114.

SOUMIS POUR :
☐ INFORMATION (F) ☐ REVUE (R)

LA REVUE NE CONSTITUE PAS L'ACCEPTATION DES DÉTAILS DE LA CONCEPTION, DES CALCULS, DES MÉTHODES D'ESSAI NI DES MATÉRIAUX MIS AU POINT OU CHOISIS PAR LE FOURNISSEUR. ELLE NE LIBÈRE PAS NON PLUS LE FOURNISSEUR DE SON OBLIGATION DE SE CONFORMER INTÉGRALEMENT À SES OBLIGATIONS PRÉVUES AU CONTRAT NI À AUCUNE AUTRE DE SES OBLIGATIONS.

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☐ 3. REVUE, POURSUIVRE LE TRAVAIL, INSÉRER LES COMMENTAIRES EN T, RE SOUMETTRE.
☐ 4. NON ACCEPTÉ. TRAVAIL INTERROMPU. REVISER ET, RE SOUMETTRE.
☐ 5. POUR INFORMATION SEULEMENT.

NOM : ______________ DATE : ______________
SIGNATURE : ______________

NOM DU PROJET : USINE D'ENRICHISSEMENT GUELB II
NOM DU FOURNISSEUR : FLSMIDTH
DESCRIPTION DU LOT : WET CONCENTRATE BUILDING
Nos D'ÉTIQUETTE :
Nos DE MODÈLE : 4010-CH-0701
CLASSIFICATION :

N° DE PROJET	N° DE BON DE COMMANDE	CODE LDRF	N° SÉQUENTIEL	N° DE SOUMISSION
020486	P-40-5004	D06	0005	01

N° DE DOCUMENT SLI

SNC·LAVALIN
SNC•LAVALIN
455, Boul. René-Lévesque Ouest
Montréal (Québec)
Canada H2Z 1Z3

CLIENT: Snim el Rhein II MAURITANIE

ISSUE RECORD

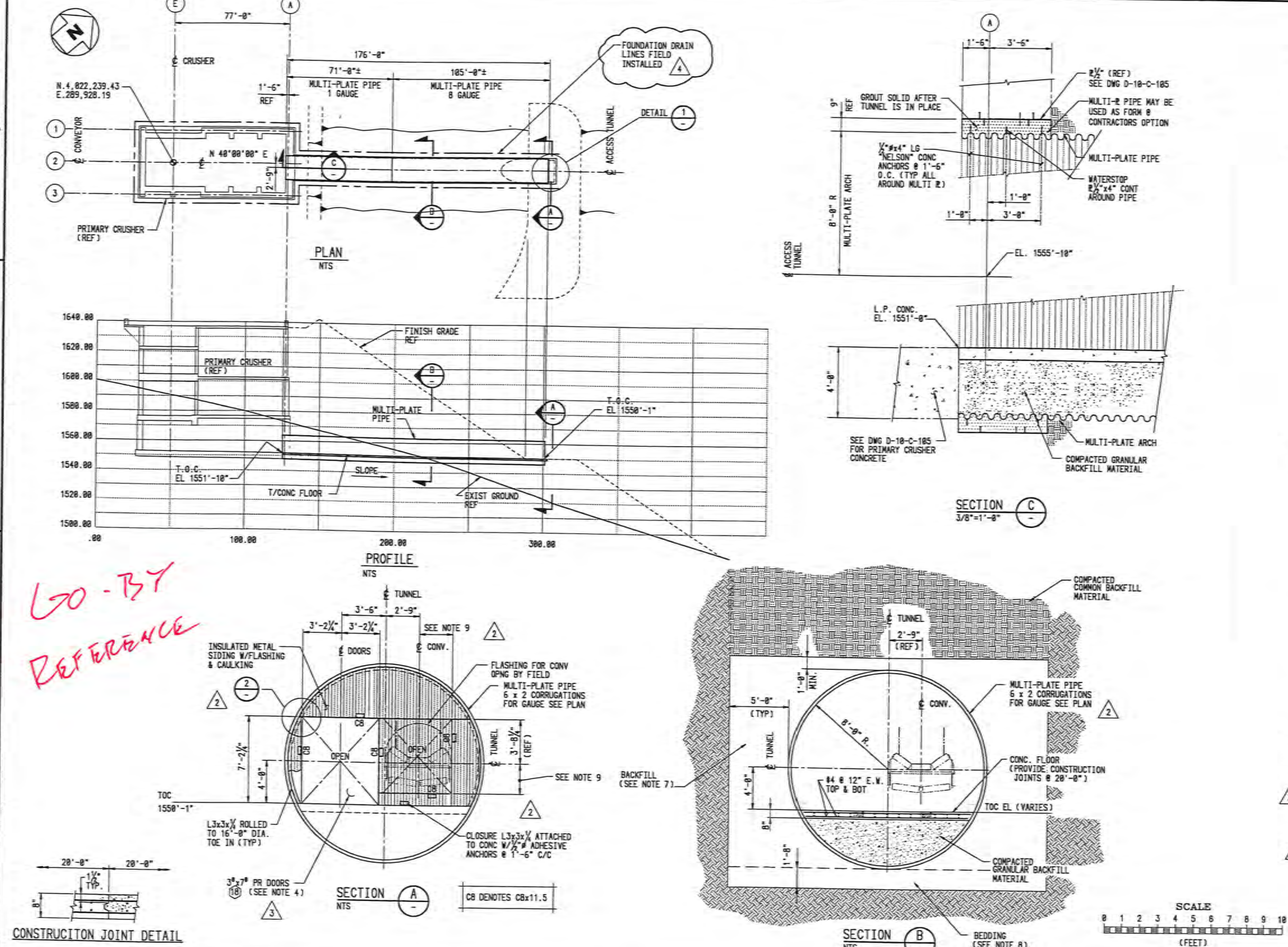
DATE	ISS	REV	TO	FOR
1	–	–	–	INITIAL RELEASE

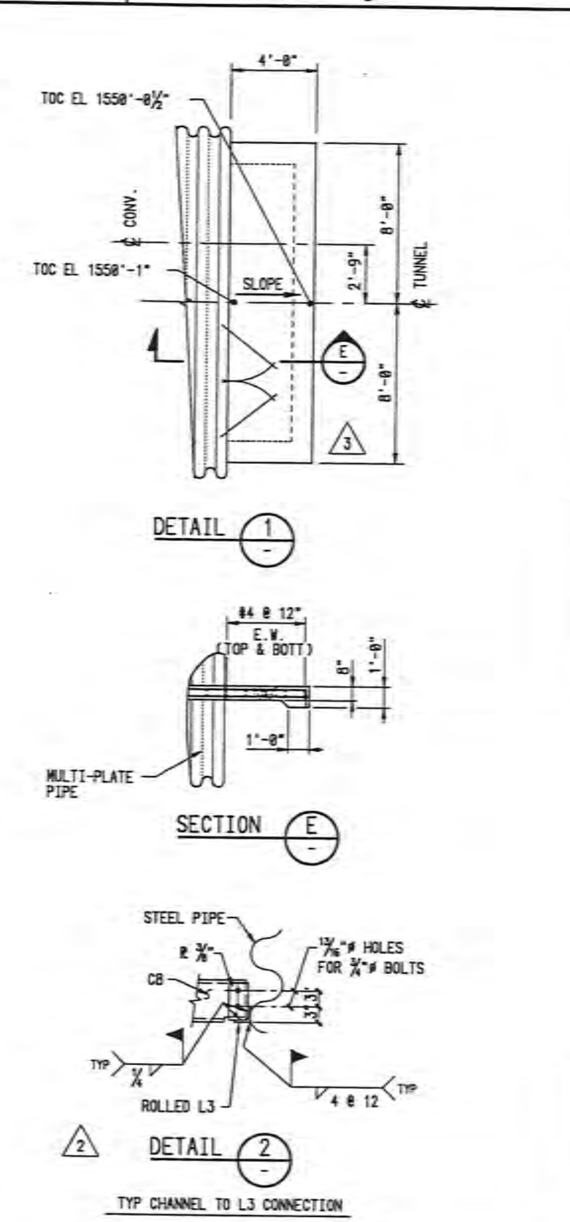
Scale	Drawn	Appr.	Appr. Date	Zone/Description	
NOTED	JEF	–	2/15/2011	Sheet Size: A1	Sheet 1 of 2

SNIM GUELB II BENEFICIATION
WET CONCENTRATE BUILDING
4010-CH-0701 CHUTE SPILE BARS
DETAILS

Mat.: NA | Weight: NA | Welds: NA

FLSMIDTH No.: DT-10082-M-115 Rev.: 1.01





NOTES:

1. FOR GENERAL CONCRETE NOTES SEE DWG D-00-S-010
2. MULTI-PLATE TUNNEL SHALL BE INSTALLED AS PER MANUFACTURERS SPECIFICATIONS
3. ALL PRIMARY CRUSHER ACCESS TUNNEL CONCRETE SHALL BE 4000 PSI
4. SEE DWG. D-26-A-019 FOR DOOR AND HARDWARE SCHEDULE.
5. SEE SPEC. #SP-64181-C-012 FOR SIDING & FLASHING.
6. FIELD LOCATE AND INSTALL ADHESIVE ANCHORS FOR CONVEYOR LEG SUPPORTS.
7. BACKFILL SHALL BE IN ACCORDANCE WITH ADOT & PF SECTION 603, PLACED IN 6" LIFTS, AND COMPACTED TO NOT LESS THAN 95% OF ASTM 1557 (MODIFIED PROCTOR).
8. BEDDING SHALL BE IN ACCORDANCE WITH ADOT & PF SECTION 603, PLACED IN 6" LIFTS, AND COMPACTED TO NOT LESS THAN 90% OF ASTM 1557 (MODIFIED PROCTOR).
9. FIELD LOCATE C8, TRIM & FLASH METAL SIDING AS REQUIRED TO MATCH CONV. COVER.

REVISIONS

NO	DESCRIPTION	DATE	BY	CHK	APP
1	REMOVED CONC. WALL @ MULTI PL. ACCESS TUNNEL	5-31-95	GVW	DSF	AJM
2	ADD PIPE BEDDING & BACK FILL DETAILS, ADD DETAIL 2	8-02-95	GLT	GB	AJM
3	ADDED DOOR SCHEDULE NUMBERS	11-27-95	SDS	DSF	AJM
4	REMOVED HOLD	1-17-96	SDS	JA	

CF822C

REFERENCE DRAWINGS

DRAWING NO	TITLE
D-50-M-001	G.A. PLOT PLAN

ISSUE RECORD

DATE	ISS	REV	TO	FOR
1-23-96	9	4	CLIENT/CONTRACTOR	CONSTRUCTION
4-04-95	2	B	CLIENT/CONTRACTOR	APPROVAL & BID
4-06-95	3	B	AK FIRE MARSHALL	BLDG. PERMIT
4-17-95	4	0	CLIENT/CONTRACTOR	CONSTRUCTION
6-01-95	5	1	CLIENT/CONTRACTOR	CONSTRUCTION
6-29-95	6	1	CEI	INTERFACE INFO
8-04-95	7	2	CLIENT/CONTRACTOR	CONSTRUCTION
11-30-95	8	3	CLIENT/CONTRACTOR	CONSTRUCTION

ENGR RECORD

	BY	DATE
DRAWN	GB	3-15-95
CHECKED	DSF	3-24-95
ELEC CHECK	CJH	3-29-95
INST CHECK	GB	3-29-95
MECH CHECK	RF	3-29-95
PIPING CHECK	DD	3-29-95
STRUCT CHECK	JBK	3-30-95
SQUAD LEADER	GB	3-30-95
CHIEF ENG/DES	GB	3-29-95
PROJECT ENG	DMW	3-29-95
CONST. REVIEW	AW	4-17-95
CLIENT APPROVAL	ATM	4-17-95

FORT KNOX
AMAX GOLD INC.
FAIRBANKS GOLD MINING, INC.
FORT KNOX GOLD PROJECT
FAIRBANKS, ALASKA

Ford, Bacon & Davis
INCORPORATED

TITLE
PRIMARY CRUSHER ACCESS TUNNEL
PLAN, SECTIONS & DETAILS
CONCRETE

FOR FAIRBANKS GOLD MINING INC. FAIRBANKS, ALASKA

SCALE	JOB NO	FBDT CAD CONTROL NO	DRAWING NO	REV
NOTED SF=32	64181	4181S0LG	D-11-C-114	4

Multi-Plate Tunnel Design

Reference: Handbook of Steel Drainage & Highway Construction Products (1994 Edition)

Tunnel Properties:

D =	16	ft	Tunnel Diameter
Corrugation =	6 x 2	inches	Corrugations
t =	10	gage	Wall Thickness
I =	0.94	in^4/ft	Moment of Inertia per ft of Tunnel (see Table 1)
A=	2.003	in^2/ft	Wall Area per ft of Tunnel (see Table 1)
r =	0.68	in	Radius of Gyration
E =	29000	ksi	Modulus of Elasticity
f_y =	33000	psi	Yield Strength
f_b =	33000	psi	Allowable Wall Stress
f_c =	16500	psi	Design Stress

Soil Properties:

H =	8	ft	Height of Cover
γ_{soil} =	110	pcf	Unit Weight of Backfill Material
ρ =	90%		Compaction Density

Loading:

DL =	880	psf	Dead Load
LL =	1500	psf	Live Load
K =	1		Load Factor (see Table 2)
P_v =	2380	psf	Design Pressure
C =	19040	plf	Ring Compression

Results:

A_{req} =	1.154	in^2/ft	**OK**	Required Wall Area per ft of Tunnel
FF =	0.016	< 0.02	**OK**	Flexibility Factor

Table 1: Properties of Corrugated Pipe

Corrugation P X D inches	Specified Gage									
	1	3	5	7	8	10	12	14	16	18
	Moment of Inertia, I, in^4 / ft of Width									
1 ½ x ¼					0.02	0.015	0.01	0.007	0.005	0.004
2 x ½					0.072	0.059	0.043	0.03	0.023	0.018
2 ½ x ½					0.069	0.054	0.041	0.029	0.023	0.018
3 x 1					0.301	0.242	0.186	0.131	0.104	0.083
5 x 1					0.301	0.244	0.188	0.133	0.106	
6 x 2	1.99	1.754	1.523	1.296	1.154	0.938	0.725			
	Cross Sectional Wall Area, in^2 / ft of Width									
1 ½ x ¼					2.093	1.712	1.331	0.95	0.761	0.608
2 x ½					2.249	1.838	1.428	1.019	0.815	0.652
2 ½ x ½					2.133	1.744	1.356	0.968	0.775	0.619
3 x 1					2.458	2.008	1.56	1.113	0.89	0.711
5 x 1					2.196	1.788	1.39	0.992	0.794	
6 x 2	4.119	3.658	3.199	2.739	2.449	2.003	1.556			

Table 2: K Value

Density	K*
70%	1.95
75%	1.3
80%	0.99
85%	0.86
90%	0.75
95%	0.65

*** if H<D, then K = 1**

Multi-Plate Tunnel Design

Reference: Handbook of Steel Drainage & Highway Construction Products (1994 Edition)

Tunnel Properties:

D =	4.5	ft	Tunnel Diameter
Corrugation =	3 x 1	inches	Corrugations
t =	14	gage	Wall Thickness
I =	0.13	in^4/ft	Moment of Inertia per ft of Tunnel (see Table 1)
A=	1.113	in^2/ft	Wall Area per ft of Tunnel (see Table 1)
r =	0.34	in	Radius of Gyration
E =	29000	ksi	Modulus of Elasticity
f_y =	33000	psi	Yield Strength
f_b =	33000	psi	Allowable Wall Stress
f_c =	16500	psi	Design Stress

Soil Properties:

H =	15	ft	Height of Cover
γ_{soil} =	105	pcf	Unit Weight of Backfill Material
ρ =	90%		Compaction Density

Loading:

DL =	1575	psf	Dead Load
LL =	1500	psf	Live Load
K =	0.75		Load Factor (see Table 2)
P_v =	2306	psf	Design Pressure
C =	5189	plf	Ring Compression

Results:

A_{req} =	0.314	in^2/ft	**OK**	Required Wall Area per ft of Tunnel
FF =	0.009	< 0.02	**OK**	Flexibility Factor

Table 1: Properties of Corrugated Pipe

Corrugation P X D inches	Specified Gage									
	1	3	5	7	8	10	12	14	16	18
	Moment of Inertia, I, in^4 / ft of Width									
1 ½ x ¼					0.02	0.015	0.01	0.007	0.005	0.004
2 x ½					0.072	0.059	0.043	0.03	0.023	0.018
2 ½ x ½					0.069	0.054	0.041	0.029	0.023	0.018
3 x 1					0.301	0.242	0.186	0.131	0.104	0.083
5 x 1					0.301	0.244	0.188	0.133	0.106	
6 x 2	1.99	1.754	1.523	1.296	1.154	0.938	0.725			
	Cross Sectional Wall Area, in^2 / ft of Width									
1 ½ x ¼					2.093	1.712	1.331	0.95	0.761	0.608
2 x ½					2.249	1.838	1.428	1.019	0.815	0.652
2 ½ x ½					2.133	1.744	1.356	0.968	0.775	0.619
3 x 1					2.458	2.008	1.56	1.113	0.89	0.711
5 x 1					2.196	1.788	1.39	0.992	0.794	
6 x 2	4.119	3.658	3.199	2.739	2.449	2.003	1.556			

Table 2: K Value

Density	K*
70%	1.95
75%	1.3
80%	0.99
85%	0.86
90%	0.75
95%	0.65

*** if H<D, then K = 1**

Multi-Plate Tunnel Design

Reference: Handbook of Steel Drainage & Highway Construction Products (1994 Edition)

Tunnel Properties:

D =	7.5	ft	Tunnel Diameter
Corrugation =	3 x 1	inches	Corrugations
t =	12	gage	Wall Thickness
I =	0.186	in^4/ft	Moment of Inertia per ft of Tunnel (see Table 1)
A=	1.56	in^2/ft	Wall Area per ft of Tunnel (see Table 1)
r =	0.34	in	Radius of Gyration
E =	29000	ksi	Modulus of Elasticity
f_y =	33000	psi	Yield Strength
f_b =	33000	psi	Allowable Wall Stress
f_c =	16500	psi	Design Stress

Soil Properties:

H =	15	ft	Height of Cover
γ_{soil} =	110	pcf	Unit Weight of Backfill Material
ρ =	90%		Compaction Density

Loading:

DL =	1650	psf	Dead Load
LL =	1500	psf	Live Load
K =	0.75		Load Factor (see Table 2)
P_v =	2363	psf	Design Pressure
C =	8859	plf	Ring Compression

Results:

A_{req} =	0.537	in^2/ft	**OK**	Required Wall Area per ft of Tunnel
FF =	0.018	< 0.02	**OK**	Flexibility Factor

Table 1: Properties of Corrugated Pipe

Corrugation P X D inches	Specified Gage									
	1	3	5	7	8	10	12	14	16	18
	Moment of Inertia, I, in^4 / ft of Width									
1 ½ x ¼					0.02	0.015	0.01	0.007	0.005	0.004
2 x ½					0.072	0.059	0.043	0.03	0.023	0.018
2 ½ x ½					0.069	0.054	0.041	0.029	0.023	0.018
3 x 1					0.301	0.242	0.186	0.131	0.104	0.083
5 x 1					0.301	0.244	0.188	0.133	0.106	
6 x 2	1.99	1.754	1.523	1.296	1.154	0.938	0.725			
	Cross Sectional Wall Area, in^2 / ft of Width									
1 ½ x ¼					2.093	1.712	1.331	0.95	0.761	0.608
2 x ½					2.249	1.838	1.428	1.019	0.815	0.652
2 ½ x ½					2.133	1.744	1.356	0.968	0.775	0.619
3 x 1					2.458	2.008	1.56	1.113	0.89	0.711
5 x 1					2.196	1.788	1.39	0.992	0.794	
6 x 2	4.119	3.658	3.199	2.739	2.449	2.003	1.556			

Table 2: K Value

Density	K*
70%	1.95
75%	1.3
80%	0.99
85%	0.86
90%	0.75
95%	0.65

*** if H<D, then K = 1**

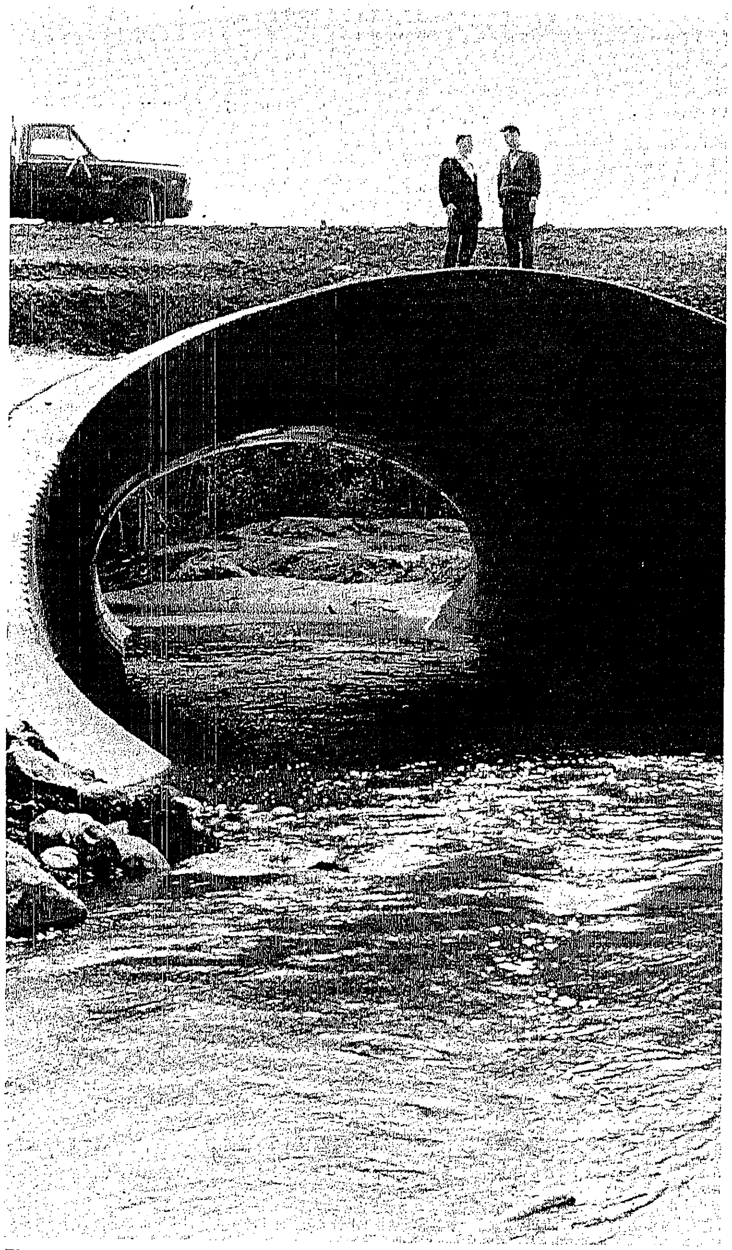


Figure 4.1 Long span corrugated steel structures are often used to replace small bridges.

Chapter 4

Structural Design

INTRODUCTION

Corrugated steel conduits, long recognized for outstanding structural strength under the heaviest of underground loadings, are now understood to be a complex composite—the result of soil and steel interaction.

Soil-steel interaction means that a flexible steel conduit acts with the surrounding soil fill to support the loads. Modern research has shown that the ideal underground structure functions to place much of the load on the soil around and over it. Corrugated steel structures approach this ideal condition.

Design methods for buried corrugated steel pipe are turning more toward the all-important soil component of the composite soil-steel system. While still conservative in reference to the soil structure, the present design procedures do formally recognize it and open the way to future developments.

RESEARCH AND DEVELOPMENT OF BURIED STRUCTURES

Earliest strength tests on corrugated steel pipe were quite crude. The tests included circus elephants balanced on unburied pipe and threshing rigs placed over shallow buried pipe.

Laboratory sand box and hydraulic tests by Talbot, Fowler and others, followed later. Fill loads were measured on buried pipe and on their foundations at Iowa State College (Marston, Spangler and others, 1913) and at the University of North Carolina (Braune, Cain, Janda) in cooperation with the *U.S. Bureau of Public Roads.*

Large-scale field tests measuring dead loads were run in 1923 on the Illinois Central Railroad at Farina, Ill. by the *American Railway Engineering Association (AREA)*. Measurements with earth pressure cells showed that flexible corrugated pipes carried only 60 percent of the 35-ft column or prism of fill above it while adjacent soil carried the remaining 40 percent of the load. These tests demonstrated for the first time that a flexible conduit and compacted earth embankment can combine to act as a composite structure. (Fig. 4.2)

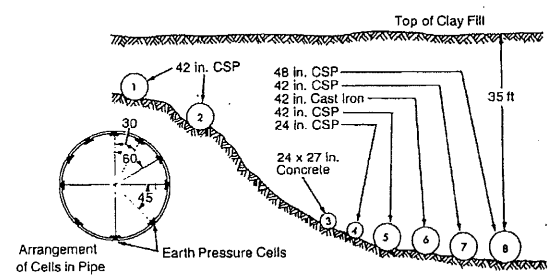


Figure 4.2 American Railway Engineering Association tests on culvert pipe at Farina, Illinois. Readings were taken on earth pressure cells.

Early efforts to rationalize the load-carrying performance of flexible conduits led to the concept of *passive side pressures* and the *Iowa Formula* for predicting deflection[1*]. Although seldom used for today's design, it has provided insight into the behavior of corrugated steel pipe.

After World War II the concept of a thin compression ring supported by soil pressures was introduced.[2,3] This fundamental concept proved compatible with experience and provided a path to rational design criteria. The national interest in blast effects from nuclear devices supplied a wealth of research and development on buried flexible structures.[4,5] This work clearly showed the potential for more efficient designs of buried corrugated steel structures. Further extensive research was sponsored by the *American Iron and Steel Institute*, 1967 to 1970, at Utah State University under the direction of Dr. Reynold K. Watkins.[6] Procedures, results and conclusions are described in condensed form in Appendix A at the end of this chapter.

California conducted a most significant research project in 1975.[7] Called the D.B. culvert, it is a 10-ft diameter structural plate pipe 0.109 in. thickness under almost 200-ft of fill. It is perhaps the only such pipe drastically underdesigned and expected to fail. The performance data from this structure contributed greatly to the development and verification of new design tools.

Since then several procedures have been developed using finite element methods: CANDE (Culvert Analysis, Design) is an FHWA (Federal Highway Administration) sponsored computer program with three levels of sophistication by M. Katona, et al.[8] SCI (Soil/Culvert Interaction) Design Method by J. M. Duncan,[9] utilizes design graphs and formulas based on finite element analysis.

FHWA Report RD77-131[10] summarizes the status of Long Span Corrugated Structures. This new family of very large structures has extended the range of corrugated steel to spans of 50 ft. Because standard design criteria was not fully applicable to long spans, special or modified design standards

* Superior numbers indicate references at the end of this chapter.

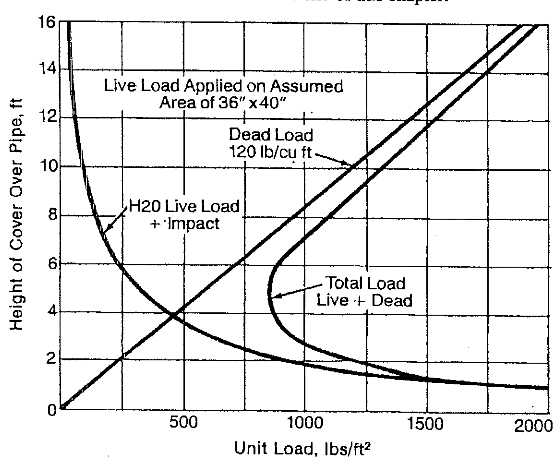


Figure 4.3 Combined H20 highway live load and dead load is a minimum at about 5 ft of cover. Live load is applied through a pavement 1 ft thick.

were established. This chapter includes the current long span criteria. Research and development on the effects of dead and live loads on the behavior of buried steel structures continues to be a subject of interest.[11-15]

LOADS ON BURIED STRUCTURES

First consideration in design is the evaluation of the loads on the conduit. Underground conduits are subject to two principal kinds of loads:

(1) dead loads developed by the embankment or trench backfill, plus stationary superimposed surface loads, uniform or concentrated; and
(2) live loads—moving loads, including impact.

LIVE LOADS

In practice, live loads on the conduit due to highway and railway traffic are related to the design of underground conduits through charts prepared by the corrugated steel pipe industry. (Figs. 4.3 and 4.4) These charts modify the theoretical distribution of live loads to values compatible with observed performance of structures under relatively light covers. Minimum cover recommendations are given in this chapter under Minimum Cover. See References 16 and 17 for a description of highway and railway live loads.

Other concentrated loads may be calculated on the basis of a load distribution slope of ½ to 1 (horizontal to vertical).

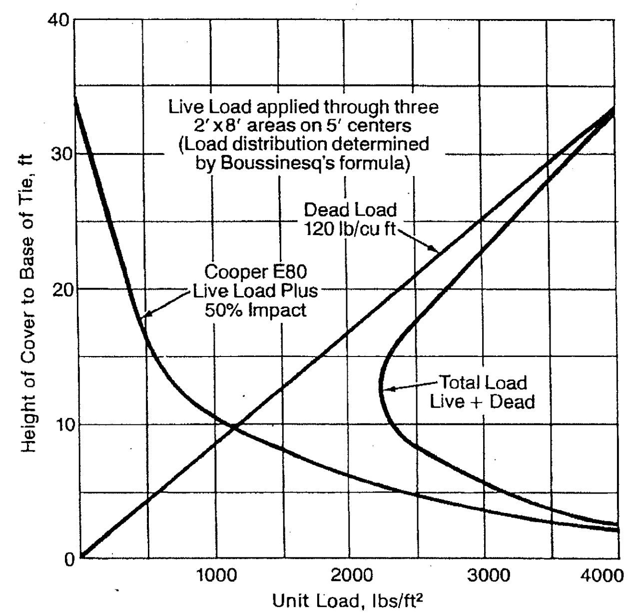


Figure 4.4 Railroad live load, Cooper E 80, combined with dead load is a minimum at about 12 ft. Live load is applied through three 2 × 8 ft areas on 5 ft centers.

AIRPORT LOADS

The significance of aircraft loads is principally in the area of required minimum cover. Some modern airport design involves very heavy wheel loads of planes not yet designed. Projected wheel configurations and weights for airplanes weighing up to 1½ to 2 million pounds have been used to develop minimum cover tables for the Federal Aviation Administration. See Tables HC-23, -24, -25, and -26, pages 273 to 275.

Table 4.1 Highway and Railway Live Loads (LL)*

Depth of Cover, Feet	Highway Loading** Load, psf H 20	Highway Loading** Load, psf H 25	Railway E 80 Loading** Depth of Cover, Feet	Railway E 80 Loading** Load, psf
1	1800	2280	2	3800
2	800	1150	5	2400
3	600	720	8	1600
4	400	470	10	1100
5	250	330	12	800
6	200	240	15	600
7	175	180	20	300
8	100	140	30	100
9	—	110	—	—

* See ASTM A 796
** Neglect live load when less than 100 psf; use dead load only.

DEAD LOADS

The dead load is considered to be the soil prism over the pipe. The unit pressure of this prism acting on the horizontal plane at the top of pipe is equal to

$$DL = w \times H \quad \text{. . . . .} \quad (1)$$

where w = Unit weight of soil, lb/ft^3
H = Height of fill over pipe, ft
DL = Dead load pressure, lb/ft^2

STRUCTURAL DESIGN OF BURIED STRUCTURES

The structural design process consists of the following:

1. Select the backfill soil density to be required or expected.
2. Calculate the design pressure.
3. Compute the compression in the pipe wall.
4. Select the allowable compressive stress.
5. Determine the thickness required.
6. Check minimum handling stiffness.
7. Check seam requirements (when applicable).
8. Check pipe-arches and arches.

1. Backfill Density

Select a percent compaction of pipe backfill for *design*. The value chosen should reflect the importance and size of the structure, and the quality that reasonably can be expected. The recommended value for routine use is 85%. This value easily will apply to ordinary installations in which most specifications will call for compaction to 90%. However, for more important structures in higher fill situations, select higher quality backfill and require the same in construction. This will extend the allowable fill height or save on thickness.

2. Design Pressure

When the height of cover is equal to or greater than the span or diameter of the structure, enter the load factor chart, Fig. 4.5, to determine the percentage of the total load acting on the steel. For routine use the 85% soil value will provide a factor of 0.86. The load factor, K, is applied to the total load to obtain the design pressure, P_v, acting on the steel. *If the height of cover is less than one pipe diameter, the total load is assumed to act on the pipe.*

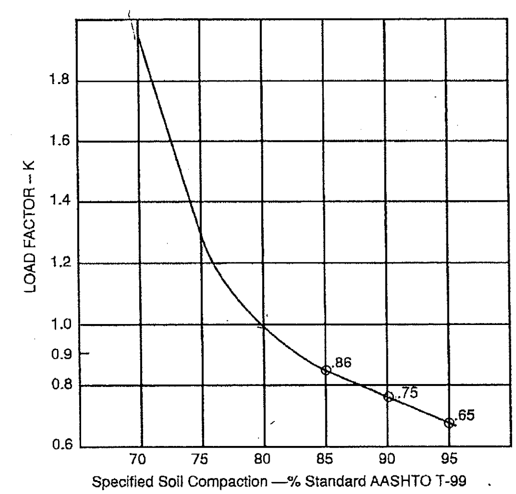


Figure 4.5 Load factors for corrugated steel pipe for backfill compacted to AASHTO T-99 density.

Increase the maximum values of FF for pipe-arch, arch and underpass shapes as follows:

Pipe-Arch FF = 1.5 × FF shown for round pipe
Arch FF = 1.5 × FF shown for round pipe
Underpass FF = same as shown for round pipe

Higher values can be used with special care or where experience has so proved. Trench condition, as in sewer design, is one example. Aluminum pipe experiences are another. For example, the flexibility factor permitted for aluminum pipe in some national specifications is more than twice that recommended above for steel. This has come about because aluminum has only one-third the stiffness of steel, the modulus for aluminum being approximately 10×10^6 psi vs 30×10^6 for steel. Where this degree of flexibility is acceptable in aluminum, it will be equally acceptable in steel.

For spiral rib pipe, a somewhat different approach is used. To obtain better control, the flexibility factors are varied with corrugation profile, sheet thickness, and type of installation, as shown in Table 4.2. The details of the installation requirements are given subsequently with the allowable fill heights in Table HC-2.

Table 4.2 Flexibility Factors for Spiral Rib Pipe

Installation Type	Flexibility Factor, in./lb.						
	¾ × 1 × 11½ Corr.			¾ × ¾ × 7½ Corr.			
	Specified Thickness, Inches			Specified Thickness, Inches			
	0.064	0.079	0.109	0.064	0.079	0.109	0.138
I	0.022	0.025	0.026	0.022	0.026	0.028	0.0368
II	0.027	0.030	0.033	0.028	0.032	0.038	0.0426
III	0.033	0.040	0.044	0.035	0.044	0.050	0.0564

Table 4.3 Moment of Inertia (I) and Cross-Sectional Area (A) of Corrugated Steel Pipe for Underground Conduits

Corrugation Pitch × Depth, inches	Specified Thickness, Inches*									
	0.052	0.064	0.079	0.109	0.138	0.168	0.188	0.218	0.249	0.280
				0.111	0.140	0.170				
	Moment of Inertia, I, $Inches^4$ per Foot of Width									
1½ × ¼	.0041	.0053	.0068	.0103	.0145	0.0196				
2 × ½	.0184	.0233	.0295	.0425	.0586	0.0719				
2⅔ × ½	.0180	.0227	.0287	.0411	.0544	0.0687				
3 × 1	.0827	.1039	.1306	.1855	.2421	0.3010				
5 × 1		.1062	.1331	.1878	.2438	0.3011				
6 × 2				.725	.938	1.154	1.296	1.523	1.754	1.990
¾ × ¾ × 7½**		.0431	.0569	.0858	0.1157					
¾ × 1 × 11½**		.0550	.0730	.1111						
	Cross-Sectional Wall Area, $Inches^2$ per Foot of Width									
1½ × ¼	.608	.761	.950	1.331	1.712	2.093				
2 × ½	.652	.815	1.019	1.428	1.838	2.249				
2⅔ × ½	.619	.775	.968	1.356	1.744	2.133				
3 × 1	.711	.890	1.113	1.560	2.008	2.458				
5 × 1		.794	.992	1.390	1.788	2.196				
6 × 2				1.556	2.003	2.449	2.739	3.199	3.658	4.119
¾ × ¾ × 7½**		.511	.715	1.192	1.729					
¾ × 1 × 11½**		.374	.524	.883						

* Where two thicknesses are shown top is corrugated steel pipe and bottom is structural plate.
** Ribbed pipe. Properties are effective values.

7. Check Longitudinal Seams

Most pipe seams develop the full yield strength of the pipe wall. However, there are exceptions in standard pipe manufacture and these are identified here. Shown below are those standard riveted and bolted seams which do not develop a strength equivalent to $f_y = 33{,}000$ psi. To maintain a consistent factor of safety of 2.0 for these pipes, it is necessary to reduce the maximum ring compression to one half the indicated seam strength. Nonstandard, or new longitudinal seam details should be checked for this same possible condition. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for these types of pipe.

Table 4.4 Ultimate Longitudinal Seam * Strengths (lb/ft)

Thickness, in.		6 × 2 in. 4 Bolts Per Ft	3 × 1 in.	2⅔ × ½ in. Rivet Seams		
Corrugated Steel Pipe	Structural Plate			5⁄16 in Single Rivet	⅜ in. Single Rivet	⅜ in. Double Rivet
0.064			28,700[1]	16,700		
0.079			35,700[1]	18,200		
0.109	0.111	42,000			23,400	
0.138	0.140	62,000	63,700[2]		24,500	49,000
0.168			70,700[2]		25,600	51,300

*See Chapter 2 for seam details.
Standard seams not shown develop full yield strength of pipe wall.
[1]Double ⅜-in. rivets.
[2]Double 7⁄16-in. rivets.

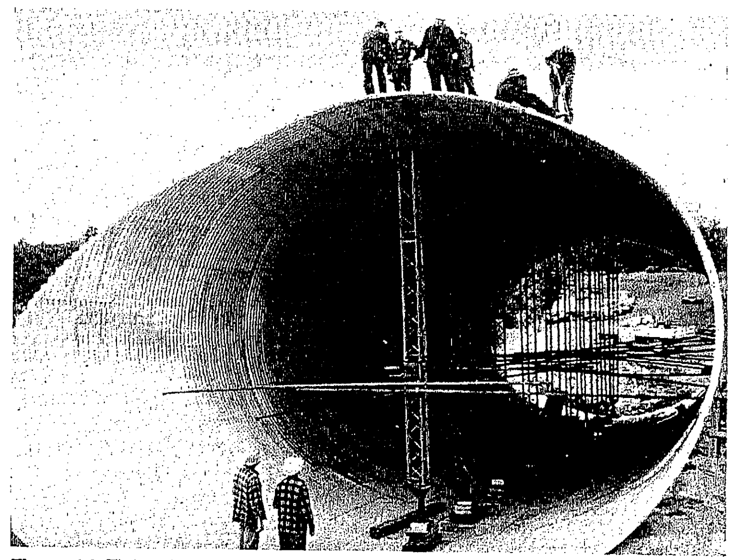


Figure 4.9 Tightening bolts on a long span structural plate culvert. Research underpass is fully instrumented prior to back filling.

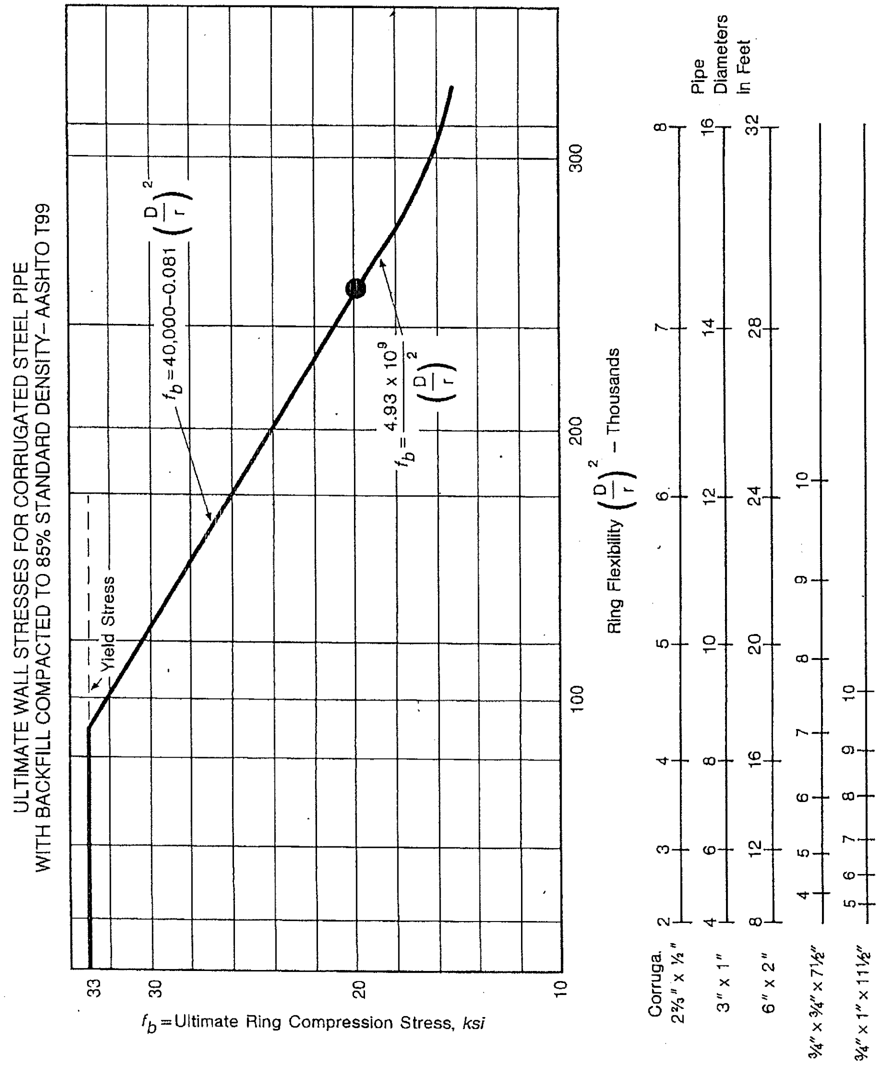


Figure 4.7 Ultimate wall or buckling stresses for corrugated steel pipe of various diameters and corrugations. The allowable stress is taken as one-half the ultimate.

5. Wall Thickness

Required wall area, A, is computed from calculated compression in the pipe wall, C, and the allowable stress, f_c.

$$A = \frac{C}{f_c} \quad (8)$$

From Table 4.3 select the wall thickness providing the required area in the same corrugation used to select the allowable stress.

6. Check Handling Stiffness

Minimum pipe stiffness requirements for practical handling and installation without undue care or bracing have been established through experience and formulated. The resultant flexibility factor, FF, limits the size of each combination of corrugation and metal thickness.

$$FF = \frac{D^2}{EI} \quad (9)$$

where: E = Modulus of elasticity = 30×10^6 psi
D = Diameter or span, in.
I = Moment of inertia of wall, in.^{4}/in.

Recommended maximum values of FF for ordinary installations:

FF = 0.0433 for factory-made pipe with riveted, welded, or helical seams
FF = 0.0200 for field-assembled pipe with bolted seams

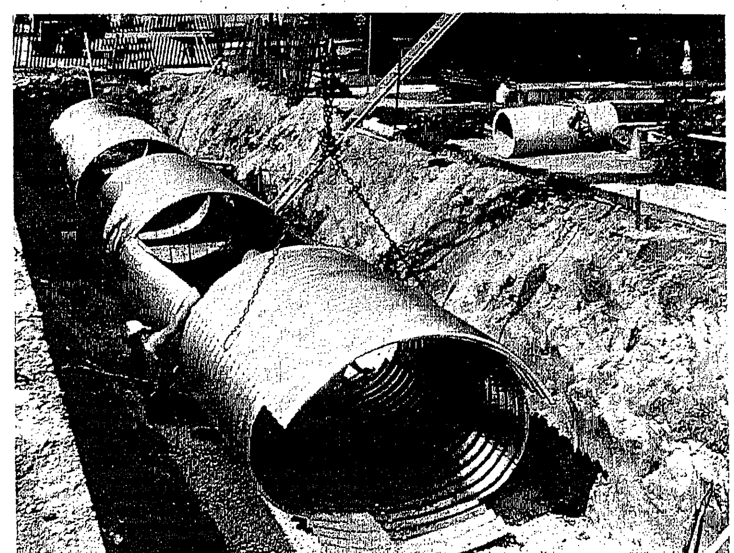


Figure 4.8 Preassembling structural plate pipe speeds installation.

Total load on pipe becomes:

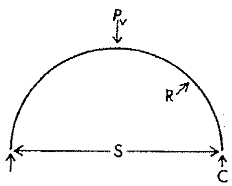
$$P_v = K \times (DL + LL), \text{ when } H \geq S \ldots\ldots \quad (2a)$$

$$P_v = (DL + LL), \text{ when } H < S \ldots\ldots \quad (2b)$$

where: P_v = Design pressure, in psf
K = Load factor
DL = Dead load, in psf
LL = Live load, in psf
H = Height of cover
S = Span

3. Ring Compression

The compressive thrust in the conduit wall is equal to the radial pressure acting on the wall multiplied by the wall radius or: $C = P \times R$. This thrust, called ring compression, is the force carried by the steel. The ring compression force acts tangentially to the conduit wall. For conventional structures in which the top arc approaches a semicircle, it is convenient to substitute half the span for the wall radius.



Then: $$C = P_v x \frac{S}{2} \ldots\ldots \quad (3)$$

where: C = Ring compression, lb/ft
P_v = Design pressure, lb/ft^2
S = Span, in ft

4. Allowable Wall Stress

The ultimate compressive stresses, f_b, for corrugated steel structures with backfill compacted to 85% standard AASHTO density and a minimum yield point of 33,000 psi, are shown in Fig. 4.7. The ultimate compression in the pipe wall is expressed by the following equations: (4), (5), (6). The first is the specified minimum yield point of the steel which represents the *zone of wall crushing* or yielding, the second represents the *interaction zone of yielding* and ring buckling, and the third represents the *ring buckling zone.*

$$f_b = f_y = 33{,}000 \text{ psi, when } \frac{D}{r} < 294 \ldots\ldots \quad (4)$$

$$f_b = 40{,}000 - .081\left(\frac{D}{r}\right)^2, \text{ when } \frac{D}{r} > 294 \text{ and} < 500 \ldots \quad (5)$$

$$f_b = \frac{4.93\times10^9}{\left(\frac{D}{r}\right)^2}, \text{ when } \frac{D}{r} > 500 \ldots\ldots \quad (6)$$

where: D = Diam. or span, in.
r = Radius of gyration, in.

A factor of safety of 2 is applied to the ultimate wall stress to obtain the design stress, f_c.

$$f_c = \frac{f_b}{2} \ldots\ldots \quad (7)$$

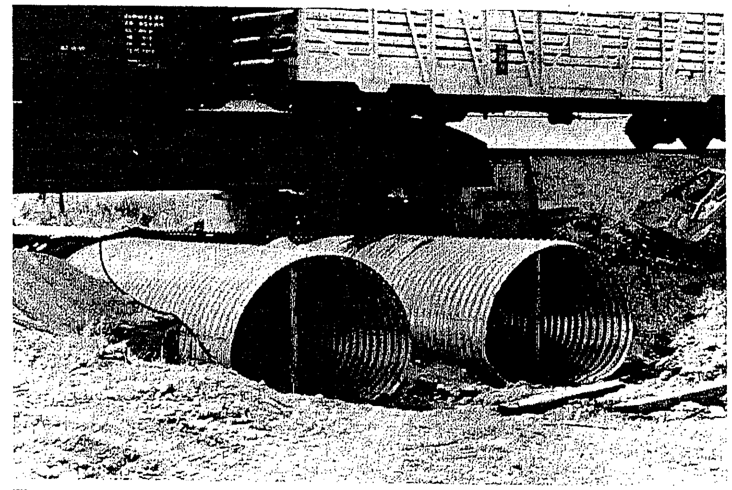


Figure 4.6 Twin corrugated steel pipe-arches are erected in place under an existing short span structure. This technique minimizes track down-time and eliminates the need for a detour.