RETROFIT OF SPLIT BRIDGE COLUMNS

Supporting Appendices

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Washington State
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Washington State Transportation Commission
Planning and Capital Program Management
in cooperation with:
U.S. DOT - Federal Highway Administration

Research Report – Supporting Appendices

Research Project 9902-26
Split Columns

RETROFIT OF SPLIT BRIDGE COLUMNS

APPENDICES

RETROFIT DESIGN CALCULATIONS

by

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APPENDIX A

FYFE COMPANY RETROFIT DESIGN CALCULATIONS

WSU COLUMN RETROFIT

Equations utilized in following spreadsheet:

Effective column height based on provided moment values and corresponding diagrams.
Plastic hinge length=(0.8)(Effective column height)+0.15(60ksi)(bar diameter).
Ideal yeild curvature is taken from the moment-curvature analysis.
Yield displacement=(ideal yeild curvature)*(effective column height)^2/3.
Neutral axis depth is taken from the moment-curvature analysis.
Mu is taken from the moment-curvature analysis.
Required ultimate curvature =(curvature ductility)(ideal yeild curvature).
Required ultimate compressive strain=(required ultimate curvature)(neutral axis depth)
Required volumetric ratio=0.8(required ultimate compressive strain-0.004)(1.5f'c)/(fu*eu)
fu=ultimate composite stress=60ksi
eu=ultimate composite strain=0.02
Required jacket thickness=0.5(required volumetric ratio)((depth*width)/(depth+width))
Required number of layers for required curvature=required thickness/0.051
Max. Shear is from a 50% overstrength value.

IN-PHASE TRANSVERSE LOADING																			
width (in.)	depth (in.)	f'c (ksi)	bar dia. (in.)	height (in.)	eff. ht.	hinge length	ductility req	curv duct.	ideal yeild curv.	yield displ. (in.)	N/A (in.)	Mu (kip-in.)	req. ult. curv.	req. ult.comp. strain	req. vol. ratio	req. jckt. thick (in.)	*req. # of layers*	max. shear (kip)	req. thick (in.)	*# lrs for shear*
7	13	5	0.375	96	96	10.155	8	24.29	2.38E-04	7.31E-01	1.2	426.6	0.0058	0.0069	0.0136	0.0308	*0.60*	6.6656	0.01494	*0.29*
16	13	5	0.375	96	96	10.155	6	17.636	2.40E-04	7.37E-01	0.8	558.6	0.0042	0.0034	-0.0028	-0.0102	*-0.20*	8.7281	0.019563	*0.38*

OUT OF PHASE LOADING																				
width (in.)	depth (in.)	f'c (ksi)	bar dia. (in.)	height (in.)	eff. ht.	hinge length	ductility req	curv. duct	ideal yeild curv.	yield displ. (in.)	N/A (in.)	Mu (kip-in.)	req. ult. curv.	req. ult.comp. strain	req. vol. ratio	req. jckt. thick (in.)	*req. # of layers*	max. shear (kip)	req. thick (in.)	*# lrs for shear*
13	7	5	0.375	60	60	7.275	6	15.633	4.26E-04	5.11E-01	0.7	156.6	0.0067	0.0047	0.0031	0.0069	*0.14*	3.915	0.016296	*0.32*

REFERENCES:

1 ICBO Acceptance Criteria for Concrete Strengthening Using Fiber-Reinforced Composite Systems, AC125, April 1997, Sect. 7.3.2.4.

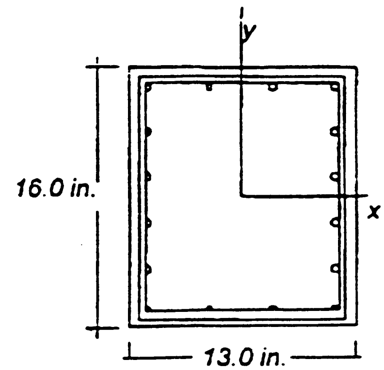
2 Priestley, M.J.N., Seible, F., Calvi, G.M., Seismic Design and Retrofit of Bridges, John Wiley & Sons, Inc., New York, 1996, pp 566-623.

3 Priestley, M.J.N., Seible, F., Seismic Retrofit of Bridge Columns Using High StrengthFiberglass/Epoxy Jackets -Design Recommendations-, Department of Applied Mechanics and Engineering Sciences, University of California San Diego, 1994

OUT - OF - PHASE

SECTION PROPERTIES:

Section Depth	*= 16.0 in.*
Section Width	*= 13.0 in.*
Cover to Main Steel	*= 0.7 in.*
Concrete Strength	*= 5.00 ksi*
Concrete Model	*= Mander*
Steel Strength	*= 40.0 ksi*
Young's Modulus	*= 29000.0 ksi*
Steel Model	*= Mild Strength Steel*
Tension Side Reinforcement	*= 4 Bars (#3 Bars)*
Compression Side Reinforcement	*= 4 Bars (#3 Bars)*
Side Reinforcement	*= 4 Bars (#3 Bars) each side*



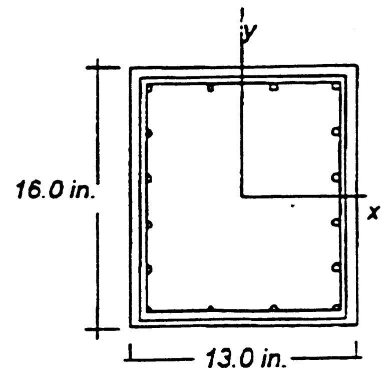
Hoop Size	*= #3 Bars*	*Average Number of Legs*	*= 2.0*
Hoop Spacing	*= 12.0 in.*	*Hoop Strength*	*= 40.0 ksi*

SECTION ANALYSIS RESULTS:

Applied Axial Load	*= 0.0 Kips*
Moment Capacity	*= 56.4 Kips ft (677.4 Kips in.)*
Section N.A. Depth	*= 1.0 in.*
Section Curvature	*= 3.95e-03 1/in.*
Maximum Concrete Strain	*= 4.0000e-03*
Extreme Steel Strain	*= 5.5818e-02*

SECTION PROPERTIES:

Section Depth	= 16.0 in.
Section Width	= 13.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Strength	= 5.00 ksi
Concrete Model	= Mander
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Steel Model	= Mild Strength Steel
Tension Side Reinforcement	= 4 Bars (#3 Bars)
Compression Side Reinforcement	= 4 Bars (#3 Bars)
Side Reinforcement	= 4 Bars (#3 Bars) each side



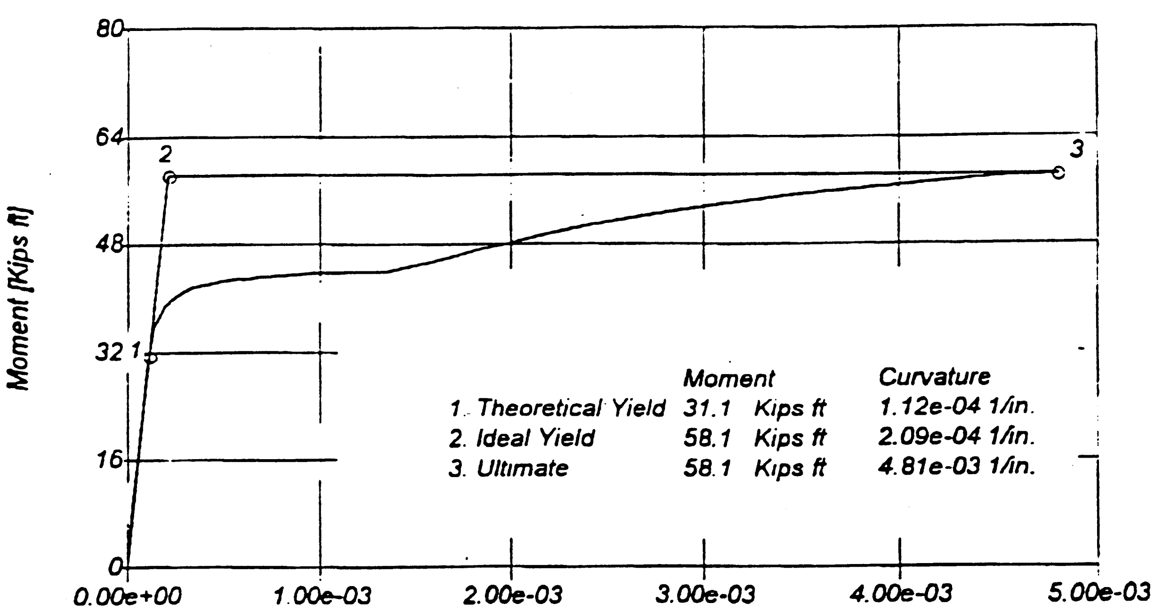
Hoop Size	= #3 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 12.0 in.	*Hoop Strength*	= 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:

The Ideal Moment Capacity is based on the concrete strain of 0.005.

Applied Axial Load	= 0.0 Kips
Eleff	= 2.32e+04 Kips sq.ft
Curvature Ductility	= 23.1 1/in.

Moment Curvature Plot

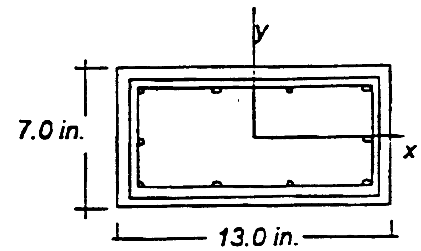


Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	3.1 in.	-0.00040	9.6 Kips ft	3.27e-05 1/in.	
0.0002	3.1 in.	-0.00080	19.2 Kips ft	6.55e-05 1/in.	
0.0003	3.1 in.	-0.00120	28.7 Kips ft	9.81e-05 1/in.	
0.0004	2.9 in.	-0.00171	35.7 Kips ft	1.38e-04 1/in.	__Yield
0.0005	2.6 in.	-0.00244	38.6 Kips ft	1.92e-04 1/in.	
0.0006	2.3 in.	-0.00333	40.3 Kips ft	2.57e-04 1/in.	
0.0007	2.1 in.	-0.00439	41.3 Kips ft	3.32e-04 1/in.	
0.0008	1.9 in.	-0.00559	42.0 Kips ft	4.17e-04 1/in.	
0.0009	1.8 in.	-0.00693	42.4 Kips ft	5.11e-04 1/in.	
0.0010	1.6 in.	-0.00839	42.8 Kips ft	6.13e-04 1/in.	
0.0011	1.5 in.	-0.00998	43.0 Kips ft	7.22e-04 1/in.	
0.0012	1.4 in.	-0.01165	43.2 Kips ft	8.38e-04 1/in.	
0.0013	1.4 in.	-0.01341	43.4 Kips ft	9.60e-04 1/in.	
0.0014	1.3 in.	-0.01523	43.5 Kips ft	1.08e-03 1/in.	
0.0015	1.2 in.	-0.01713	43.6 Kips ft	1.22e-03 1/in.	
0.0016	1.2 in.	-0.01906	43.7 Kips ft	1.35e-03 1/in.	
0.0017	1.2 in.	-0.02082	44.4 Kips ft	1.47e-03 1/in.	
0.0018	1.1 in.	-0.02251	45.2 Kips ft	1.59e-03 1/in.	
0.0019	1.1 in.	-0.02422	46.1 Kips ft	1.70e-03 1/in.	
0.0020	1.1 in.	-0.02592	46.9 Kips ft	1.82e-03 1/in.	
0.0025	1.0 in.	-0.03415	50.4 Kips ft	2.39e-03 1/in.	
0.0030	1.0 in.	-0.04191	53.0 Kips ft	2.93e-03 1/in.	
0.0035	1.0 in.	-0.04934	54.9 Kips ft	3.45e-03 1/in.	
0.0040	1.0 in.	-0.05656	56.4 Kips ft	3.95e-03 1/in.	
0.0045	1.0 in.	-0.06328	57.5 Kips ft	4.42e-03 1/in.	
0.0050	1.0 in.	-0.06874	58.1 Kips ft	4.81e-03 1/in.	

Ultimate concrete strain was exceeded.

SECTION PROPERTIES:

Section Depth	= 7.0 in.
Section Width	= 13.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Strength	= 5.00 ksi
Concrete Model	= Mander
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Steel Model	= Mild Strength Steel
Tension Side Reinforcement	= 4 Bars (#3 Bars)
Compression Side Reinforcement	= 4 Bars (#3 Bars)
Side Reinforcement	= 1 Bars (#3 Bars) each side



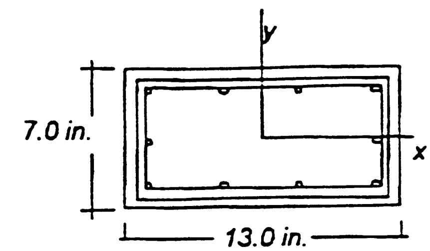
Hoop Size	= #3 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 12.0 in.	*Hoop Strength*	= 40.0 ksi

SECTION ANALYSIS RESULTS:

Applied Axial Load	= 0.0 Kips
Moment Capacity	= 13.0 Kips ft (156.6 Kips in.)
Section N.A. Depth	= 0.7 in.
Section Curvature	= 5.74e-03 1/in.
Maximum Concrete Strain	= 4.0000e-03
Extreme Steel Strain	= 3.1231e-02

SECTION PROPERTIES:

Section Depth	= 7.0 in.
Section Width	= 13.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Strength	= 5.00 ksi
Concrete Model	= Mander
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Steel Model	= Mild Strength Steel
Tension Side Reinforcement	= 4 Bars (#3 Bars)
Compression Side Reinforcement	= 4 Bars (#3 Bars)
Side Reinforcement	= 1 Bars (#3 Bars) each side



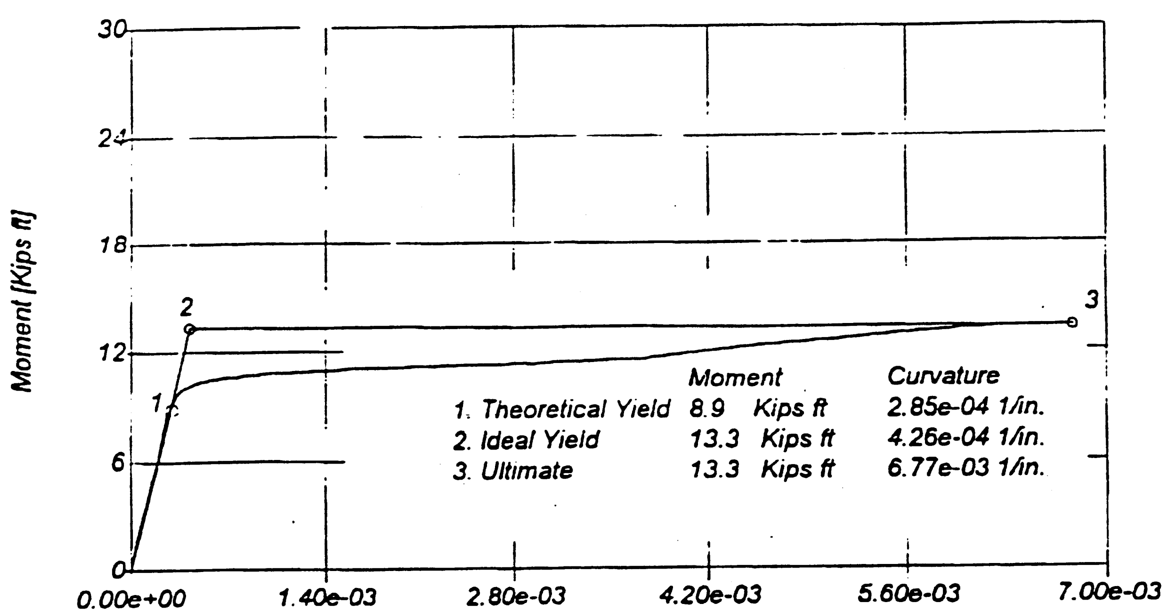
Hoop Size	= #3 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 12.0 in.	*Hoop Strength*	= 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:

The Ideal Moment Capacity is based on the concrete strain of 0.005.

Applied Axial Load	= 0.0 Kips
EIeff	= 2.60e+03 Kips sq.ft
Curvature Ductility	= 15.9 1/in.

Moment Curvature Plot



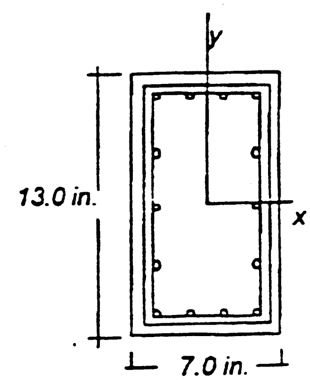
Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	1.5 in.	-0.00031	2.1 Kips ft	6.55e-05 1/in.	
0.0002	1.5 in.	-0.00063	4.3 Kips ft	1.31e-04 1/in.	
0.0003	1.5 in.	-0.00094	6.4 Kips ft	1.96e-04 1/in.	
0.0004	1.5 in.	-0.00125	8.5 Kips ft	2.61e-04 1/in.	
0.0005	1.4 in.	-0.00171	9.9 Kips ft	3.49e-04 1/in.	__Yield
0.0006	1.3 in.	-0.00238	10.3 Kips ft	4.71e-04 1/in.	
0.0007	1.1 in.	-0.00316	10.5 Kips ft	6.10e-04 1/in.	
0.0008	1.0 in.	-0.00403	10.7 Kips ft	7.63e-04 1/in.	
0.0009	1.0 in.	-0.00497	10.8 Kips ft	9.27e-04 1/in.	
0.0010	0.9 in.	-0.00596	10.9 Kips ft	1.10e-03 1/in.	
0.0011	0.9 in.	-0.00698	11.0 Kips ft	1.28e-03 1/in.	
0.0012	0.8 in.	-0.00803	11.0 Kips ft	1.46e-03 1/in.	
0.0013	0.8 in.	-0.00910	11.1 Kips ft	1.64e-03 1/in.	
0.0014	0.8 in.	-0.01017	11.1 Kips ft	1.83e-03 1/in.	
0.0015	0.7 in.	-0.01125	11.2 Kips ft	2.01e-03 1/in.	
0.0016	0.7 in.	-0.01232	11.2 Kips ft	2.20e-03 1/in.	
0.0017	0.7 in.	-0.01339	11.2 Kips ft	2.38e-03 1/in.	
0.0018	0.7 in.	-0.01444	11.3 Kips ft	2.57e-03 1/in.	
0.0019	0.7 in.	-0.01548	11.3 Kips ft	2.75e-03 1/in.	
0.0020	0.7 in.	-0.01650	11.3 Kips ft	2.92e-03 1/in.	
0.0025	0.7 in.	-0.02120	11.6 Kips ft	3.74e-03 1/in.	
0.0030	0.7 in.	-0.02517	12.3 Kips ft	4.45e-03 1/in.	
0.0035	0.7 in.	-0.02885	12.7 Kips ft	5.11e-03 1/in.	
0.0040	0.7 in.	-0.03231	13.0 Kips ft	5.74e-03 1/in.	
0.0045	0.7 in.	-0.03562	13.3 Kips ft	6.34e-03 1/in.	
0.0050	0.7 in.	-0.03785	13.3 Kips ft	6.77e-03 1/in.	

Ultimate concrete strain was exceeded.

TRANSVERSE

SECTION PROPERTIES:

Section Depth	*= 13.0 in.*
Section Width	*= 7.0 in.*
Cover to Main Steel	*= 0.7 in.*
Concrete Strength	*= 5.00 ksi*
Concrete Model	*= Mander*
Steel Strength	*= 40.0 ksi*
Young's Modulus	*= 29000.0 ksi*
Steel Model	*= Mild Strength Steel*
Tension Side Reinforcement	*= 4 Bars (#3 Bars)*
Compression Side Reinforcement	*= 4 Bars (#3 Bars)*
Side Reinforcement	*= 3 Bars (#3 Bars) each side*



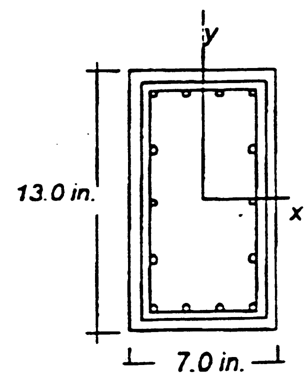
Hoop Size	*= #3 Bars*	*Average Number of Legs*	*= 2.0*
Hoop Spacing	*= 12.0 in.*	*Hoop Strength*	*= 40.0 ksi*

SECTION ANALYSIS RESULTS:

Applied Axial Load	*= 0.0 Kips*
Moment Capacity	*= 35.5 Kips ft (426.6 Kips in.)*
Section N.A. Depth	*= 1.2 in.*
Section Curvature	*= 3.37e-03 1/in.*
Maximum Concrete Strain	*= 4.0000e-03*
Extreme Steel Strain	*= 3.6907e-02*

SECTION PROPERTIES:

Section Depth	= 13.0 in.
Section Width	= 7.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Strength	= 5.00 ksi
Concrete Model	= Mander
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Steel Model	= Mild Strength Steel
Tension Side Reinforcement	= 4 Bars (#3 Bars)
Compression Side Reinforcement	= 4 Bars (#3 Bars)
Side Reinforcement	= 3 Bars (#3 Bars) each side



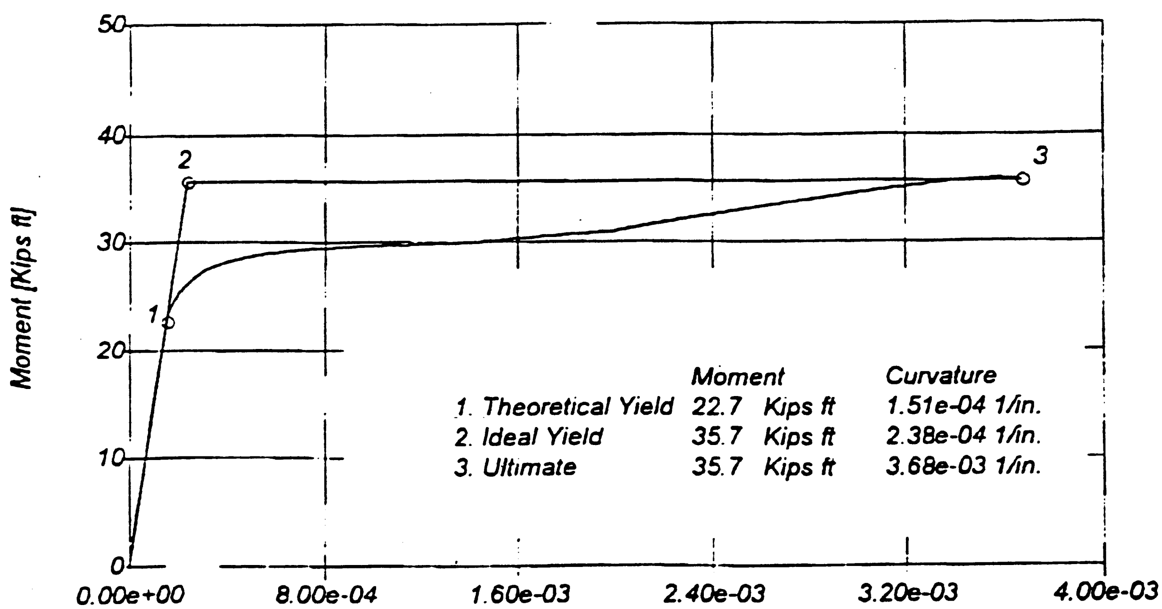
Hoop Size	= #3 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 12.0 in.	*Hoop Strength*	= 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:

The Ideal Moment Capacity is based on the concrete strain of 0.005.

Applied Axial Load	= 0.0 Kips
EIeff	= 1.25e+04 Kips sq.ft
Curvature Ductility	= 15.5 1/in.

Moment Curvature Plot



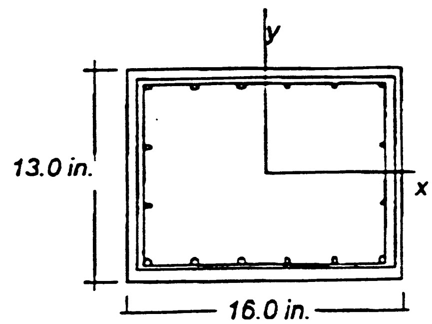
	Moment	Curvature
1. Theoretical Yield	22.7 Kips ft	1.51e-04 1/in.
2. Ideal Yield	35.7 Kips ft	2.38e-04 1/in.
3. Ultimate	35.7 Kips ft	3.68e-03 1/in.

Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	3.2 in.	-0.00028	4.8 Kips ft	3.12e-05 1/in.	
0.0002	3.2 in.	-0.00057	9.5 Kips ft	6.24e-05 1/in.	
0.0003	3.2 in.	-0.00085	14.3 Kips ft	9.36e-05 1/in.	
0.0004	3.2 in.	-0.00114	19.0 Kips ft	1.25e-04 1/in.	
0.0005	3.2 in.	-0.00143	23.5 Kips ft	1.57e-04 1/in.	__Yield
0.0006	2.9 in.	-0.00191	25.5 Kips ft	2.04e-04 1/in.	
0.0007	2.7 in.	-0.00249	26.7 Kips ft	2.58e-04 1/in.	
0.0008	2.5 in.	-0.00315	27.6 Kips ft	3.20e-04 1/in.	
0.0009	2.3 in.	-0.00390	28.2 Kips ft	3.89e-04 1/in.	
0.0010	2.2 in.	-0.00473	28.6 Kips ft	4.65e-04 1/in.	
0.0011	2.0 in.	-0.00564	29.0 Kips ft	5.46e-04 1/in.	
0.0012	1.9 in.	-0.00661	29.2 Kips ft	6.33e-04 1/in.	
0.0013	1.8 in.	-0.00763	29.4 Kips ft	7.24e-04 1/in.	
0.0014	1.7 in.	-0.00871	29.6 Kips ft	8.20e-04 1/in.	
0.0015	1.6 in.	-0.00984	29.7 Kips ft	9.20e-04 1/in.	
0.0016	1.6 in.	-0.01101	29.8 Kips ft	1.02e-03 1/in.	
0.0017	1.5 in.	-0.01219	29.9 Kips ft	1.13e-03 1/in.	
0.0018	1.5 in.	-0.01343	29.9 Kips ft	1.24e-03 1/in.	
0.0019	1.4 in.	-0.01467	30.0 Kips ft	1.34e-03 1/in.	
0.0020	1.4 in.	-0.01592	30.0 Kips ft	1.45e-03 1/in.	
0.0025	1.3 in.	-0.02197	31.0 Kips ft	1.98e-03 1/in.	
0.0030	1.2 in.	-0.02734	32.8 Kips ft	2.46e-03 1/in.	
0.0035	1.2 in.	-0.03266	34.4 Kips ft	2.93e-03 1/in.	
0.0040	1.2 in.	-0.03754	35.5 Kips ft	3.37e-03 1/in.	
0.0045	1.3 in.	-0.03981	35.9 Kips ft	3.59e-03 1/in.	
0.0050	1.4 in.	-0.04041	35.7 Kips ft	3.68e-03 1/in.	

Ultimate concrete strain was exceeded.

SECTION PROPERTIES:

Section Depth	= 13.0 in.
Section Width	= 16.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Strength	= 5.00 ksi
Concrete Model	= Mander
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Steel Model	= Mild Strength Steel
Tension Side Reinforcement	= 6 Bars (#3 Bars)
Compression Side Reinforcement	= 6 Bars (#3 Bars)
Side Reinforcement	= 2 Bars (#3 Bars) each side



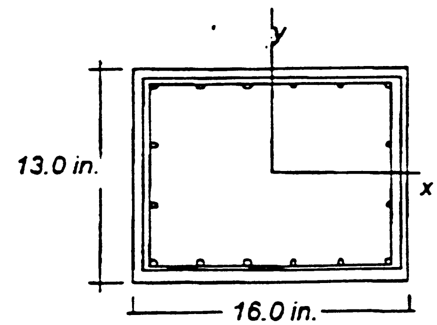
Hoop Size	= #3 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 12.0 in.	*Hoop Strength*	= 40.0 ksi

SECTION ANALYSIS RESULTS:

Applied Axial Load	= 0.0 Kips
Moment Capacity	= 46.6 Kips ft (558.6 Kips in.)
Section N.A. Depth	= 0.8 in.
Section Curvature	= 4.86e-03 1/in.
Maximum Concrete Strain	= 4.0000e-03
Extreme Steel Strain	= 5.4985e-02

SECTION PROPERTIES:

Section Depth	*= 13.0 in.*
Section Width	*= 16.0 in.*
Cover to Main Steel	*= 0.7 in.*
Concrete Strength	*= 5.00 ksi*
Concrete Model	*= Mander*
Steel Strength	*= 40.0 ksi*
Young's Modulus	*= 29000.0 ksi*
Steel Model	*= Mild Strength Steel*
Tension Side Reinforcement	*= 6 Bars (#3 Bars)*
Compression Side Reinforcement	*= 6 Bars (#3 Bars)*
Side Reinforcement	*= 2 Bars (#3 Bars) each side*



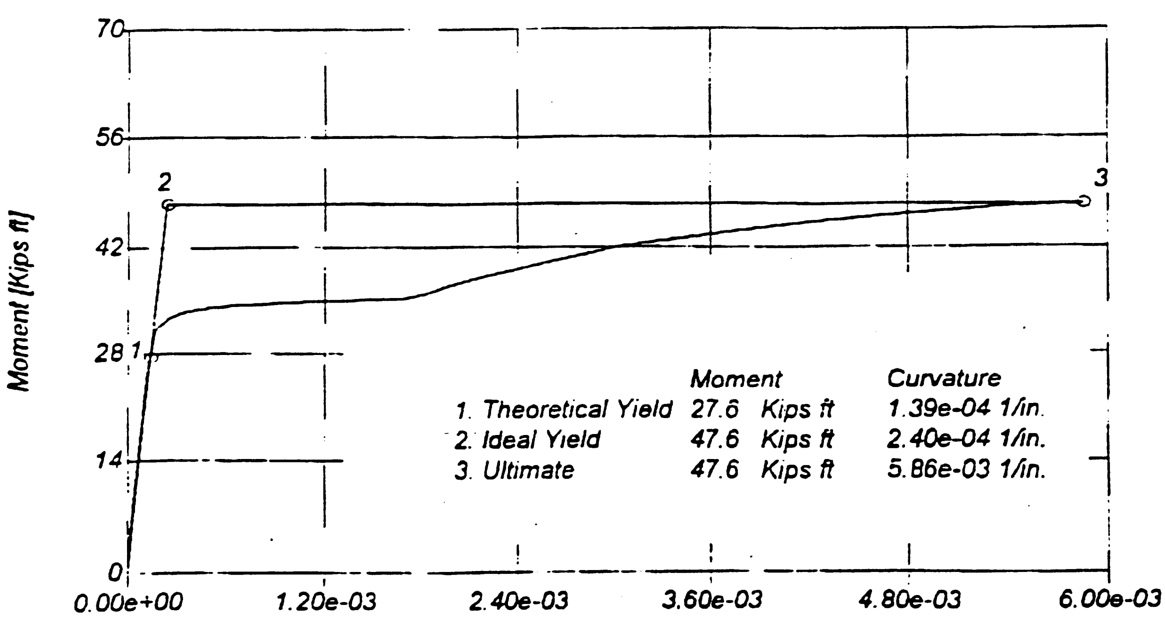
Hoop Size	*= #3 Bars*	*Average Number of Legs*	*= 2.0*
Hoop Spacing	*= 12.0 in.*	*Hoop Strength*	*= 40.0 ksi*

MOMENT CURVATURE ANALYSIS RESULTS:

The Ideal Moment Capacity is based on the concrete strain of 0.005.

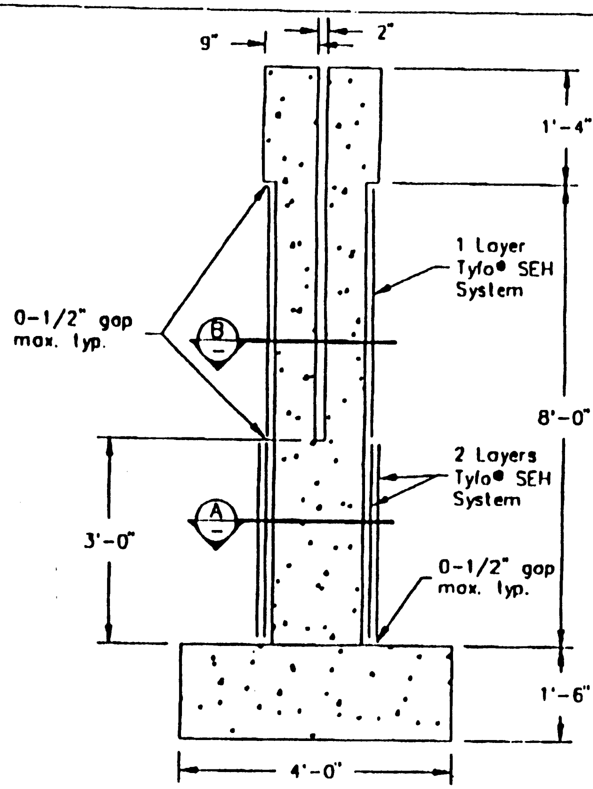
Applied Axial Load	*= 0.0 Kips*
EIeff	*= 1.65e+04 Kips sq.ft*
Curvature Ductility	*= 24.4 1/in.*

Moment Curvature Plot

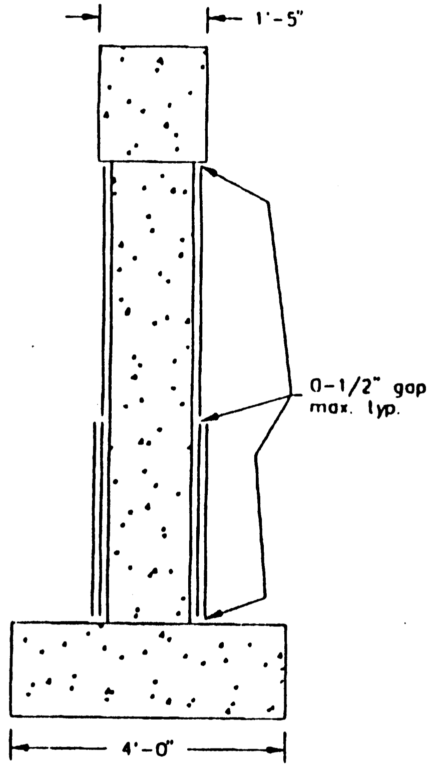


Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	2.5 in.	-0.00040	8 6 Kips ft	4.02e-05 1/in.	
0.0002	2.5 in.	-0.00079	17.1 Kips ft	8.04e-05 1/in.	
0.0003	2.5 in.	-0.00119	25.6 Kips ft	1.20e-04 1/in.	
0.0004	2.3 in.	-0.00174	31.3 Kips ft	1.74e-04 1/in.	__Yield
0.0005	2.0 in.	-0.00263	32.8 Kips ft	2.54e-04 1/in.	
0.0006	1.7 in.	-0.00372	33.6 Kips ft	3.51e-04 1/in.	
0.0007	1.5 in.	-0.00500	34.0 Kips ft	4.62e-04 1/in.	
0.0008	1.4 in.	-0.00643	34.4 Kips ft	5.87e-04 1/in.	
0.0009	1.2 in.	-0.00801	34.6 Kips ft	7.23e-04 1/in.	
0.0010	1.2 in.	-0.00970	34.7 Kips ft	8.68e-04 1/in.	
0.0011	1.1 in.	-0.01149	34.8 Kips ft	1.02e-03 1/in.	
0.0012	1.0 in.	-0.01333	34.9 Kips ft	1.18e-03 1/in.	
0.0013	1.0 in.	-0.01523	35.0 Kips ft	1.34e-03 1/in.	
0.0014	0.9 in.	-0.01715	35.1 Kips ft	1.50e-03 1/in.	
0.0015	0.9 in.	-0.01908	35.2 Kips ft	1.67e-03 1/in.	
0.0016	0.9 in.	-0.02079	35.9 Kips ft	1.82e-03 1/in.	
0.0017	0.9 in.	-0.02242	36.8 Kips ft	1.96e-03 1/in.	
0.0018	0.9 in.	-0.02404	37.6 Kips ft	2.10e-03 1/in.	
0.0019	0.9 in.	-0.02564	38.4 Kips ft	2.23e-03 1/in.	
0.0020	0.8 in.	-0.02722	39.1 Kips ft	2.37e-03 1/in.	
0.0025	0.8 in.	-0.03489	42.0 Kips ft	3.03e-03 1/in.	
0.0030	0.8 in.	-0.04220	44.0 Kips ft	3.67e-03 1/in.	
0.0035	0.8 in.	-0.04917	45.5 Kips ft	4.27e-03 1/in.	
0.0040	0.8 in.	-0.05590	46.6 Kips ft	4.86e-03 1/in.	
0.0045	0.8 in.	-0.06227	47.3 Kips ft	5.42e-03 1/in.	
0.0050	0.9 in.	-0.06721	47.6 Kips ft	5.86e-03 1/in.	

Ultimate concrete strain was exceeded.



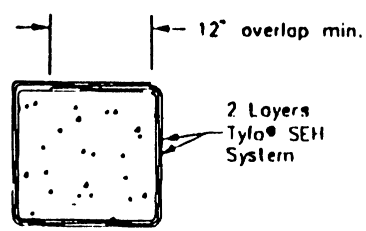
Column Split View - Elevation
Not to Scale



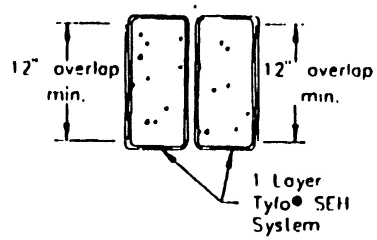
Column Side View - Elevation
Not to Scale

General Notes:

1. Ensure a 3/4" min. radius at column edges.
2. Broom-clean column surfaces
3. Apply primecoat Tylo® WS Epoxy.
4. Apply specified layers of Tylo® SEH System



(A) Section
Not to Scale



(B) Section
Not to Scale

WSDOT Research Project
Column Design
Out-of-Plane Loading

FYFE CO. LLC
6044 Cornerstone Ct. West, Suite C, San Diego, CA 92121-4730
TEL: 619-642-0694 FAX: 619-642-0947

Dwg No: 404-1	Job No 298RD	Acad File No FC\298RD\404-1
Date: 10/8/98	Drawn by E. Morrow	Chk'd by P B Milligan
Scale N.T.S.	App'd by P B Milligan	Sheet 1 of 2

APPENDIX B

SUMITOMO RETROFIT DESIGN CALCULATIONS

BERGER/ABAM
ENGINEERS INC.

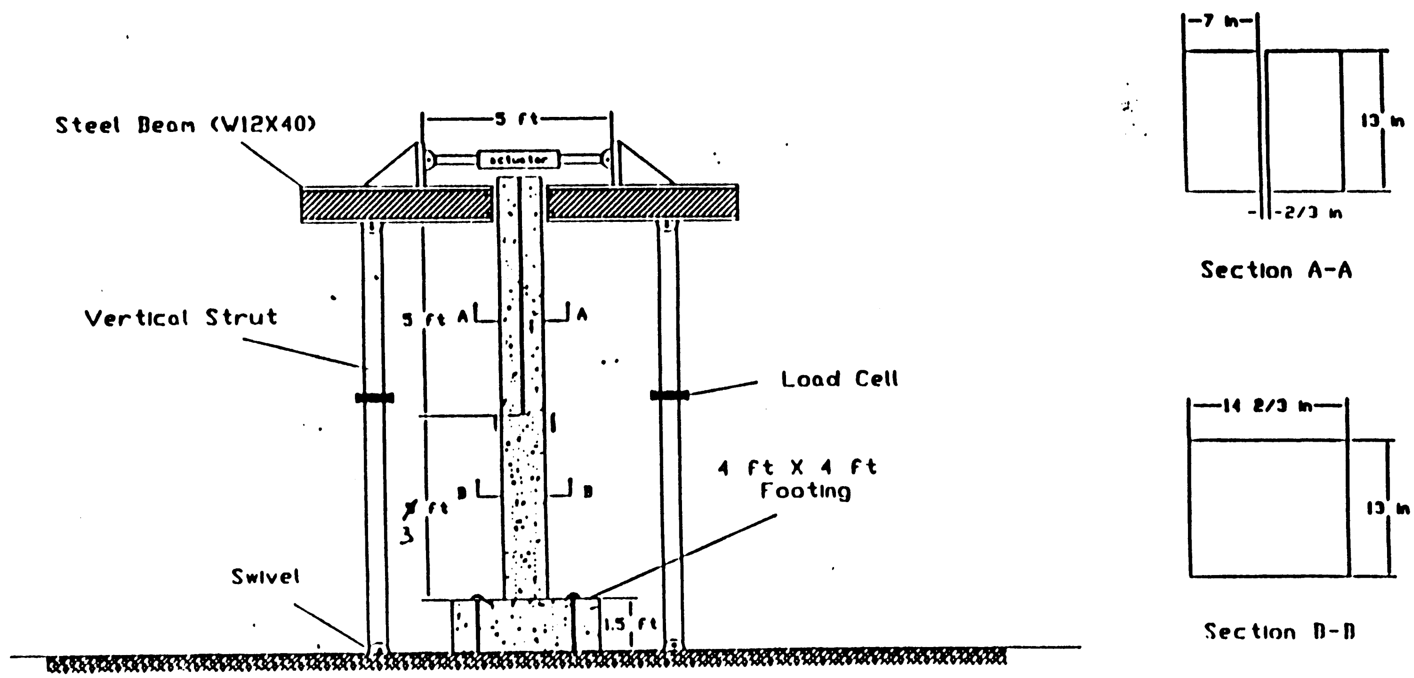
Project ______ Sheet 1 of ______
Job Number ______
Subject ______ Designer ______
Date ______

'OUT-OF-PHASE'

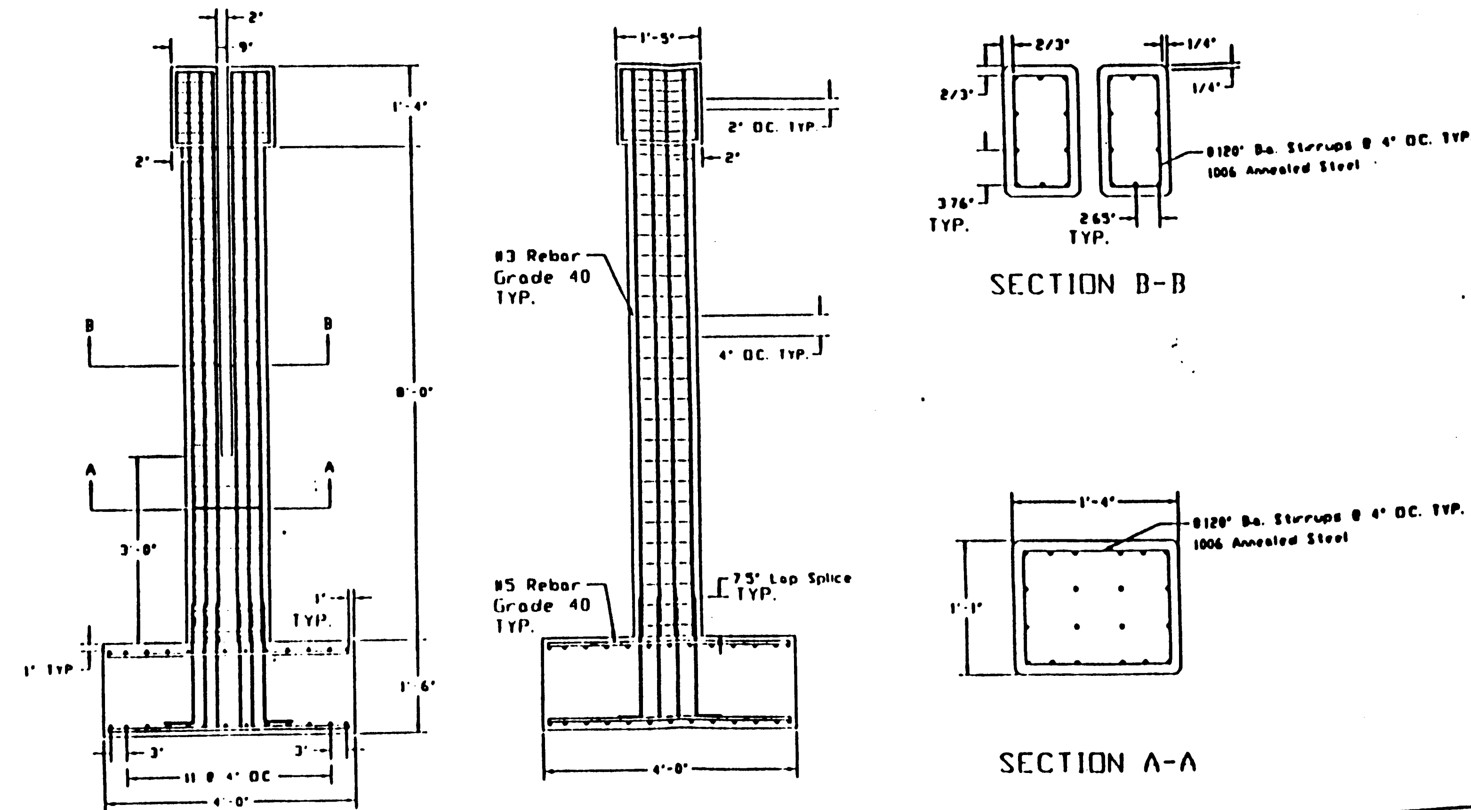
SPECIMEN DETAILS

& TEST SETUP

rg-001-061085



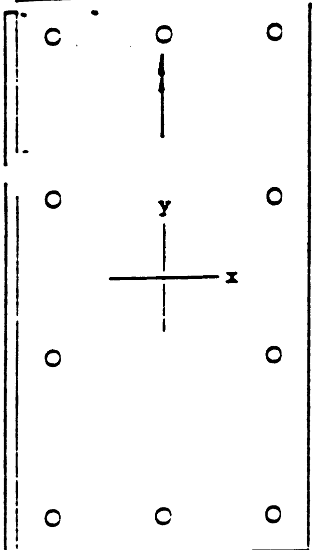
Test setup for out-of-phase Loading



Washington State University Department of Civil Engineering	Column Design - Out-of-Phase Loading WSDOT Research Project	DWG. 1

4

5



7.0 x 13.0 inch

f'c = 4.0 ksi

fy = 44.0 ksi

Confinement: Other

clr cover = 0.67 in

spacing = 2.27 in

10-#3 at 1.21%

= 1 in^2

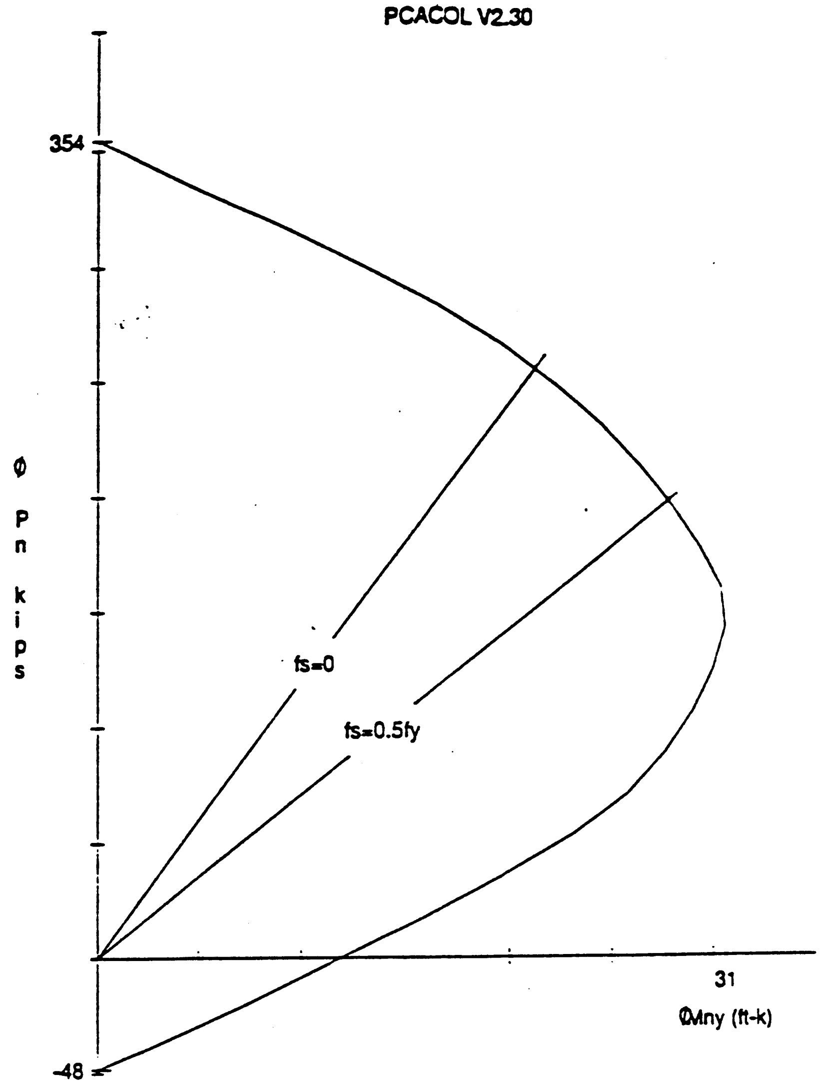
Ix = 1282 in^4

Iy = 372 in^4

Xo = 0.00 in

Yo = 0.00 in

PCACOL V2.30



Licensed To: Licensee name not yet specified.

File name: O:\1997\A97106\DOT-SP-1\GAO\OUTPHASE.COL

Project: DOT Split Column Retrof

Column Id: out-of-phase

Engineer: YQG

Date: 12/16/98 Time: 10:24:59

Code: ACI 318-89

Units: in-lb

Material Properties:

Ec = 3834 ksi eu = 0.003 in/in

fc = 3.40 ksi Es = 29000 ksi

Beta1 = 0.85

Stress Profile: Block

phi(c) = 1.00, phi(b) = 1.00

Y-axis slenderness is not considered.

```
OOOOOOO    OOOOO    OOOOO    OOOOO    OOOOO   OO
OO     OO OO    OO OO    OO OO    OO OO    OO OO
OO     OO OO    OO OO    OO OO    OO OO    OO OO
OO     OO OO       OO    OO OO       OO    OO OO
OO     OO OO       OOOOOOOO OO       OO    OO OO
OOOOOOO   OO    OO OO    OO OO    OO OO    OO OO
OO        OO    OO OO    OO OO    OO OO    OO OO
OO         OOOOO   OO    OO  OOOOO    OOOOO   OOOOO  (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections

---

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General Information:
====================

```
File Name:  O:\1997\A97106\DOT-SP-1\GAO\OUTPHASE.COL
Project:    DOT Split Column Retrof  Code: ACI 318-89
Column:     out-of-phase             Units: US in-lbs
Engineer:   YQG                      Date: 12/16/98  Time: 10:24:59

Run Option: Investigation            Short (nonslender) column
Run Axis:   Y-axis                   Column Type: Structural
```

Material Properties:
====================

```
f'c   = 4 ksi                        fy   = 44 ksi
Ec    = 3834.25 ksi                  Es   = 29000 ksi
fc    = 3.4 ksi                      erup = 0 in/in
eu    = 0.003 in/in
Stress Profile: Block                Beta1 = 0.85
```

Geometry:
=========

```
Rectangular: Width = 7 in            Depth = 13 in

Gross section area, Ag =   91 in^2
Ix =  1281.58 in^4                   Xo =  0 in
Iy =  371.583 in^4                   Yo =  0 in
```

Reinforcement:
==============

Rebar Database: ASTM

Size	Diam	Area	Size	Diam	Area	Size	Diam	Area
3	0.38	0.11	4	0.50	0.20	5	0.63	0.31
6	0.75	0.44	7	0.88	0.60	8	1.00	0.79
9	1.13	1.00	10	1.27	1.27	11	1.41	1.56
14	1.69	2.25	18	2.26	4.00			

Confinement: User-defined; phi(c) = 1, phi(b) = 1, a = 1
#3 ties with #10 bars, #4 with larger bars.

Layout: Rectangular
Pattern: Sides Different [Cover to longitudinal reinforcement]

Total steel area, As = 1.10 in^2 at 1.21%

	Top	Bottom	Left	Right
Bars	3 -# 3	3 -# 3	2 -# 3	2 -# 3
Cover(in)	0.67	0.67	0.67	0.67

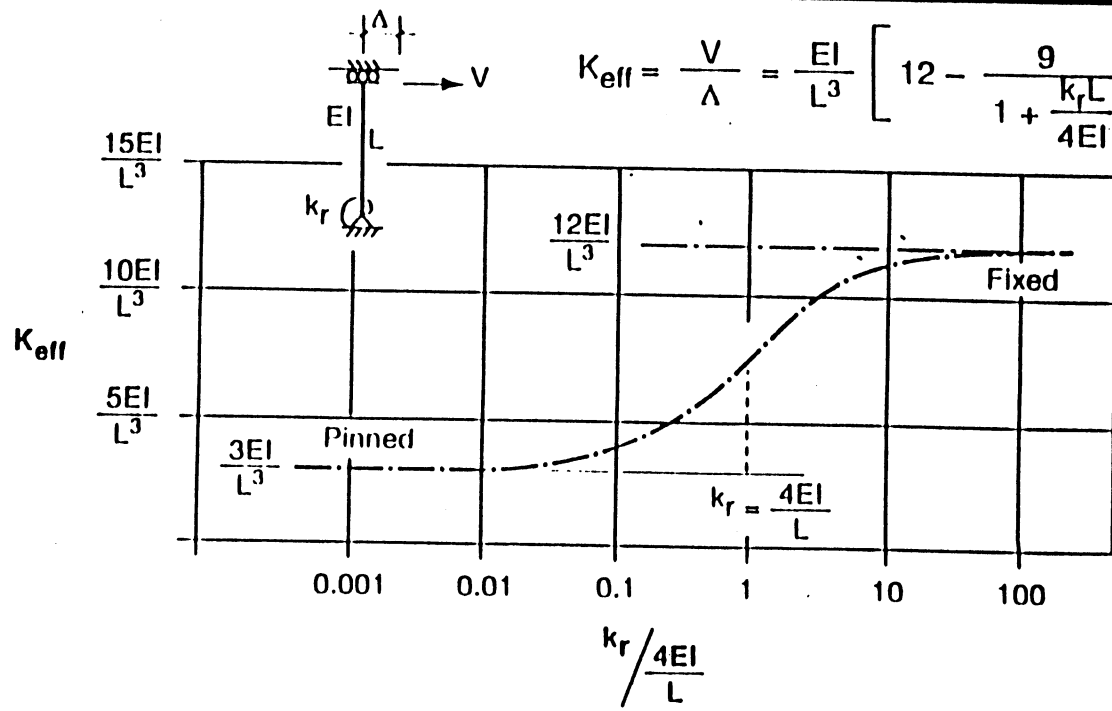
 8

Bending about		Load, P (kips)	X-Mom. (ft-k)	Y-Mom. (ft-k)	N.A. depth (in)
Y	Pure Comp.	354	0	0	12.43
	Balanced	154	0	31	4.08
	Pure Bend.	-0	0	12	0.82

Program completed as requested!

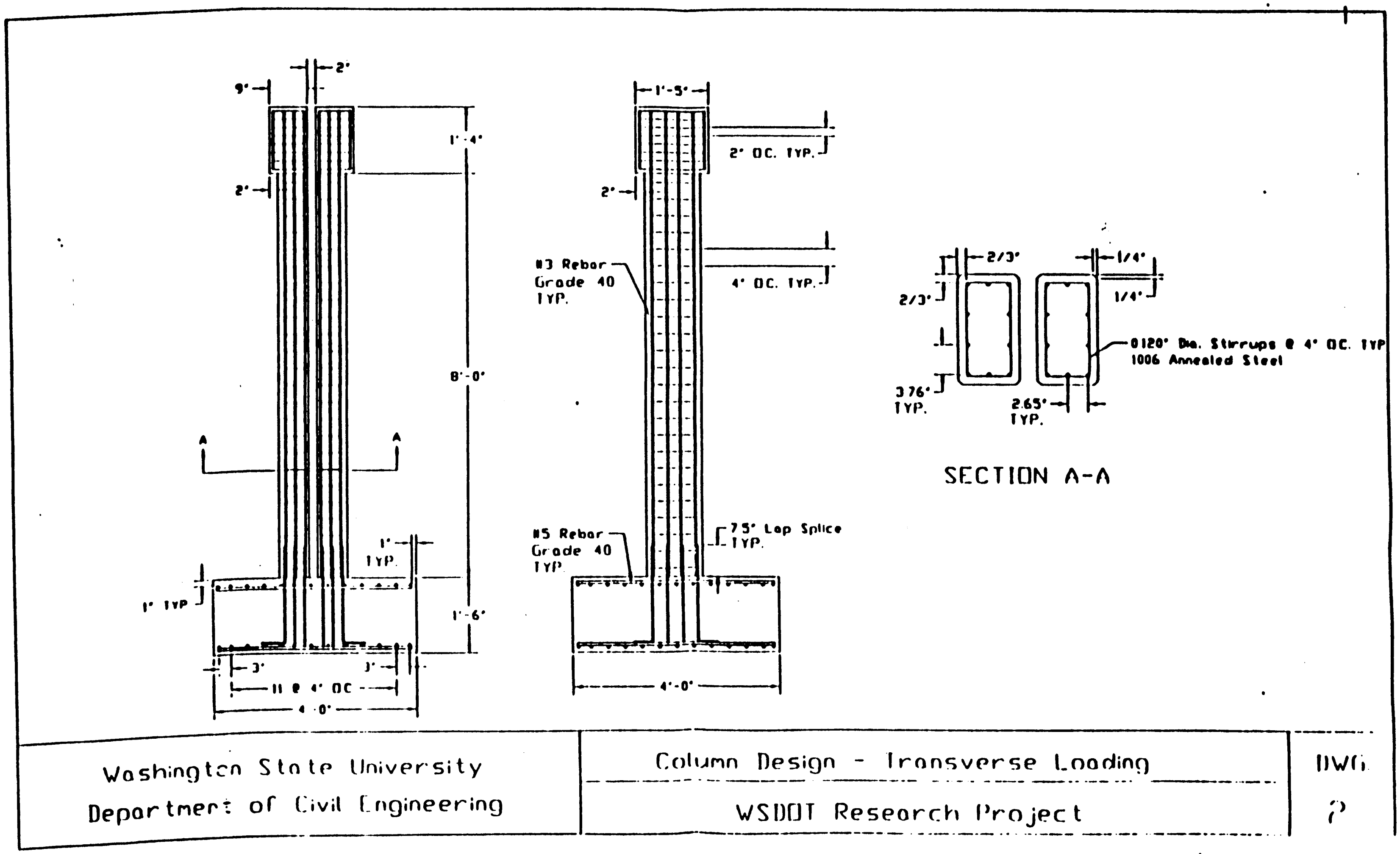
Rotational Flexibility / Fixed or Not?

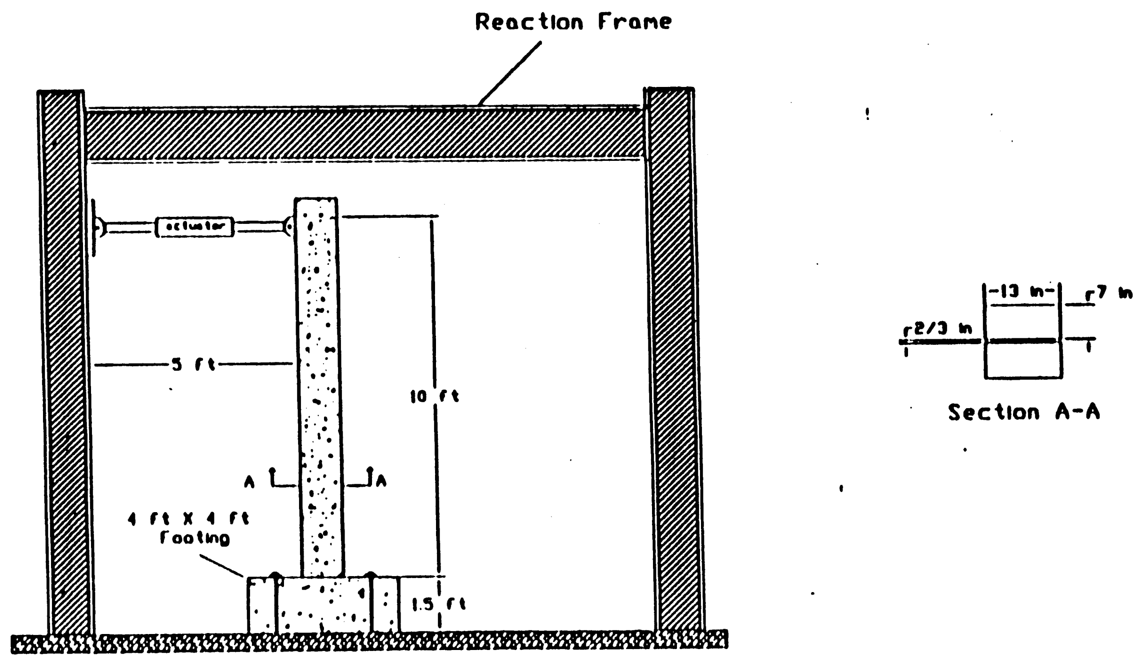
$$K_{eff} = \frac{V}{\Lambda} = \frac{EI}{L^3}\left[12 - \frac{9}{1 + \frac{k_r L}{4EI}}\right]$$



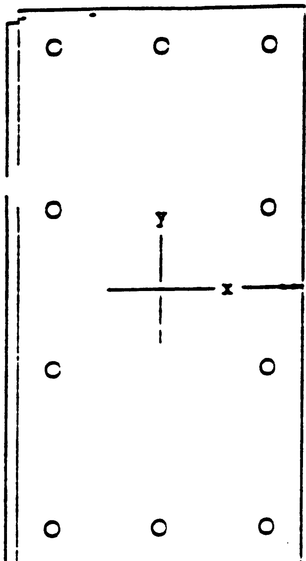
17

Session 1 Page 29 of 53
UMD-ITV
Seismic Bridge Design Applications
25 July 1996, NHI Course Code No. 13063

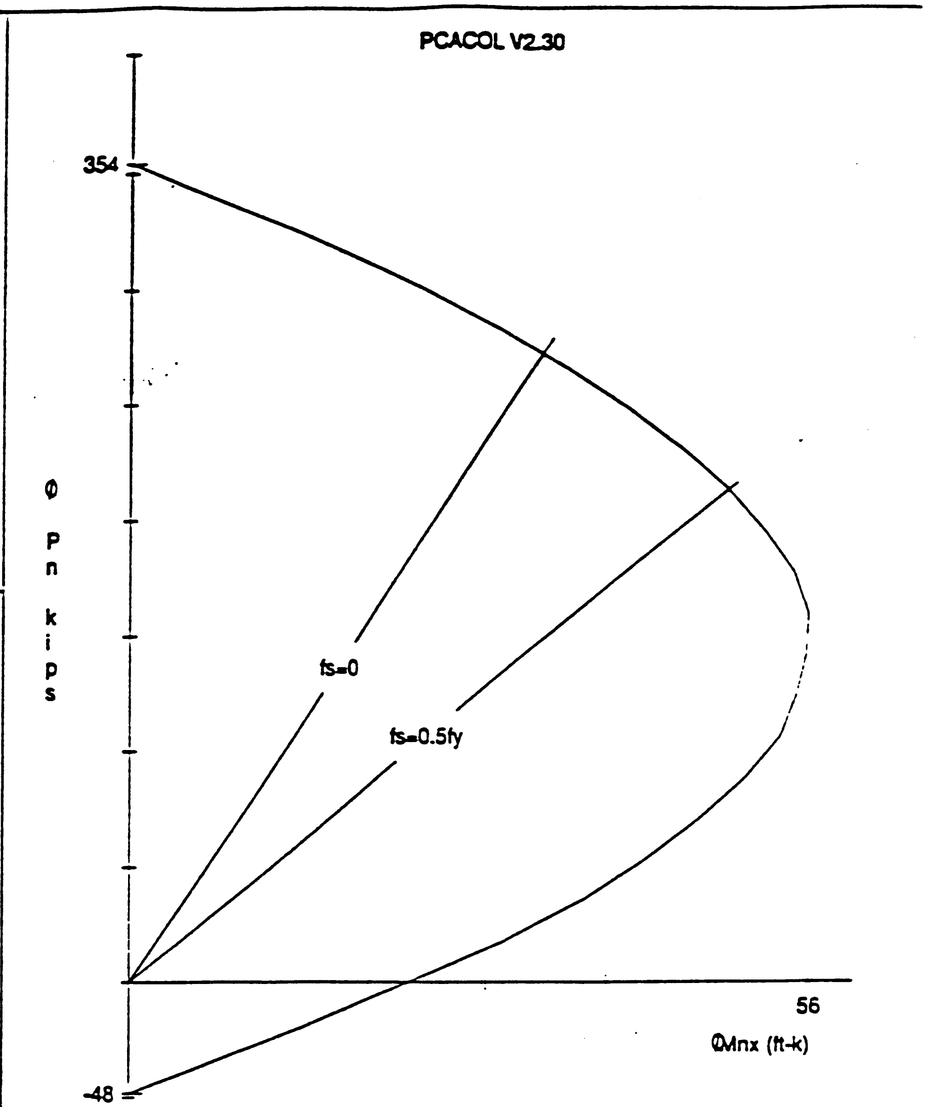

9"
2"
2"
1'-4"
8'-0"
1'-6"
A
A
1" TYP
1"
TYP.
3"
3"
11 @ 4" OC
4'-0"
1'-5"
2" OC. TYP.
4" OC. TYP.
#3 Rebar
Grade 40
TYP.
#5 Rebar
Grade 40
TYP.
7.5" Lap Splice
TYP.
4'-0"
2/3"
2/3"
1/4"
1/4"
3.76"
TYP.
2.65"
TYP.
0.120" Dia. Stirrups @ 4" OC. TYP
1006 Annealed Steel
SECTION A-A
Washington State University
Department of Civil Engineering
Column Design - Transverse Loading
WSDOT Research Project
DWG.
2



Test setup for transverse loading



7.0 x 13.0 inch



f'c = 4.0 ksi

fy = 44.0 ksi

Confinement: Other

clr cover = 0.67 in

spacing = 2.27 in

10-#3 at 1.21%

= 1 in^2

Ix = 1282 in^4

Iy = 372 in^4

Xo = 0.00 in

Yo = 0.00 in

File name: O:\1997\A97106\DOT-SP-1\GAO\TRANSV.COL

Project: DOT Split Column Retrof	Material Properties:
Column Id: Transverse	Ec = 3834 ksi eu = 0.003 in/in
Engineer: YQG	fc = 3.40 ksi Es = 29000 ksi
Date: 12/16/98 Time: 10:24:59	Beta1 = 0.85
Code: ACI 318-89	Stress Profile: Block
Units: in-lb	phi(c) = 1.00, phi(b) = 1.00

X-axis slenderness is not considered.

```
OOOOOOO    OOOOO     OOOOO     OOOOO     OOOOO    OO
OO     OO OO     OO OO     OO OO     OO OO     OO OO
OO     OO OO     OO OO     OO OO     OO OO     OO OO
OO     OO OO        OO     OO OO        OO     OO OO
OO     OO OO        OOOOOOOOO OO        OO     OO OO
OOOOOOO   OO     OO OO     OO OO     OO OO     OO OO
OO        OO     OO OO     OO OO     OO OO     OO OO
OO         OOOOO    OO     OO  OOOOO     OOOOO    OOOOO (TM)
```

Computer program for the Strength Design of Reinforced Concrete Sections
----

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General Information:
====================

File Name: O:\1997\A97106\DOT-SP-1\GAO\TRANSV.COL
Project: DOT Split Column Retrof Code: ACI 318-89
Column: Transverse Units: US in-lbs
Engineer: YQG Date: 12/16/98 Time: 10:24:59

Run Option: Investigation Short (nonslender) column
Run Axis: X-axis Column Type: Structural

Material Properties:
====================

f'c = 4 ksi fy = 44 ksi
Ec = 3834.25 ksi Es = 29000 ksi
fc = 3.4 ksi erup = 0 in/in
eu = 0.003 in/in
Stress Profile: Block Beta1 = 0.85

Geometry:
=========

Rectangular: Width = 7 in Depth = 13 in

Gross section area, Ag = 91 in^2
Ix = 1281.58 in^4 Xo = 0 in
Iy = 371.583 in^4 Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM

Size	Diam	Area	Size	Diam	Area	Size	Diam	Area
3	0.38	0.11	4	0.50	0.20	5	0.63	0.31
6	0.75	0.44	7	0.88	0.60	8	1.00	0.79
9	1.13	1.00	10	1.27	1.27	11	1.41	1.56
14	1.69	2.25	18	2.26	4.00			

Confinement: User-defined; phi(c) = 1, phi(b) = 1, a = 1
#3 ties with #10 bars, #4 with larger bars.

Layout: Rectangular
Pattern: Sides Different (Cover to longitudinal reinforcement)

Total steel area, As = 1.10 in^2 at 1.21%

	Top	Bottom	Left	Right
Bars	3 - # 3	3 - # 3	2 - # 3	2 - # 3
Cover(in)	0.67	0.67	0.67	0.67

Bending about		Load, P (kips)	X-Mom. (ft-k)	Y-Mom. (ft-k)	N.A. depth (in)
X	Pure Comp.	354	-0	-0	24.57
	Balanced	169	56	-0	8.06
	Pure Bend.	0	23	-0	1.27

Program completed as requested!

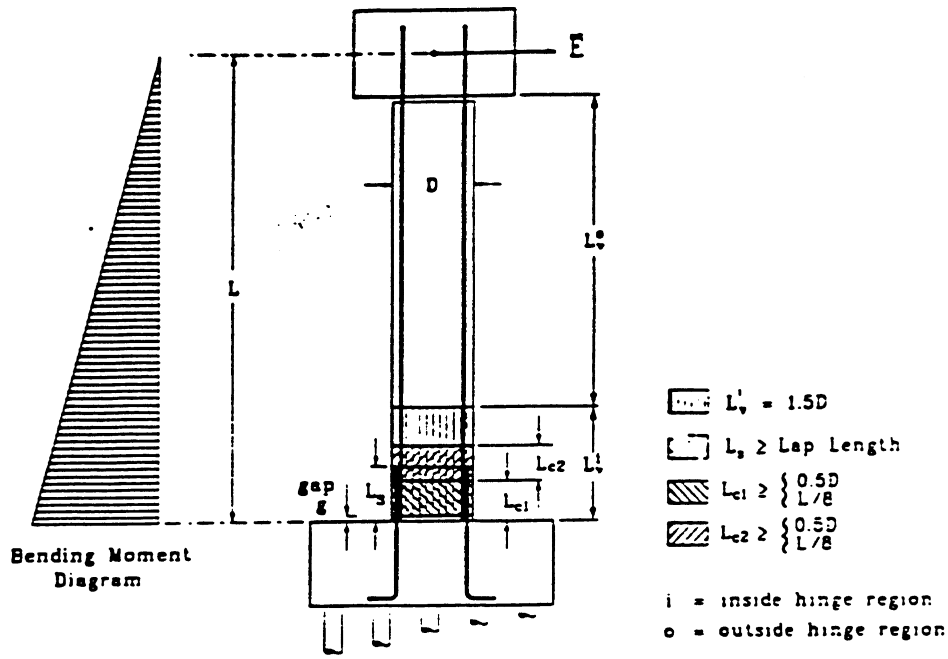


FIG 5. Carbon Jacket Regions for Bridge Column Retrofit. Single Bending

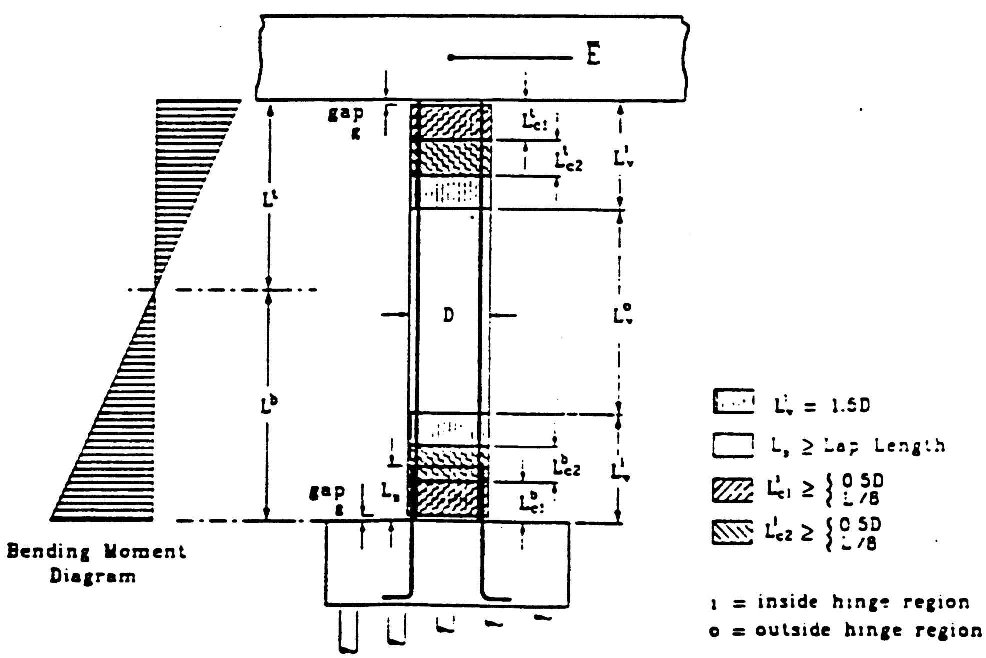


FIG 6. Carbon Jacket Regions for Bridge Column Retrofit, Double Bending

51

DRAWINGS

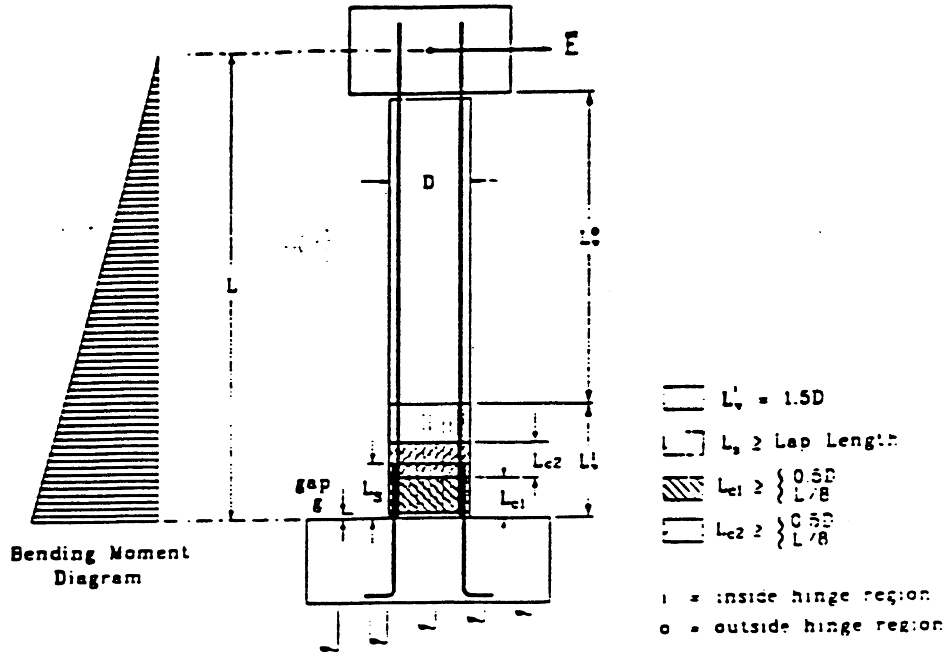


FIG 5. Carbon Jacket Regions for Bridge Column Retrofit. Single Bending

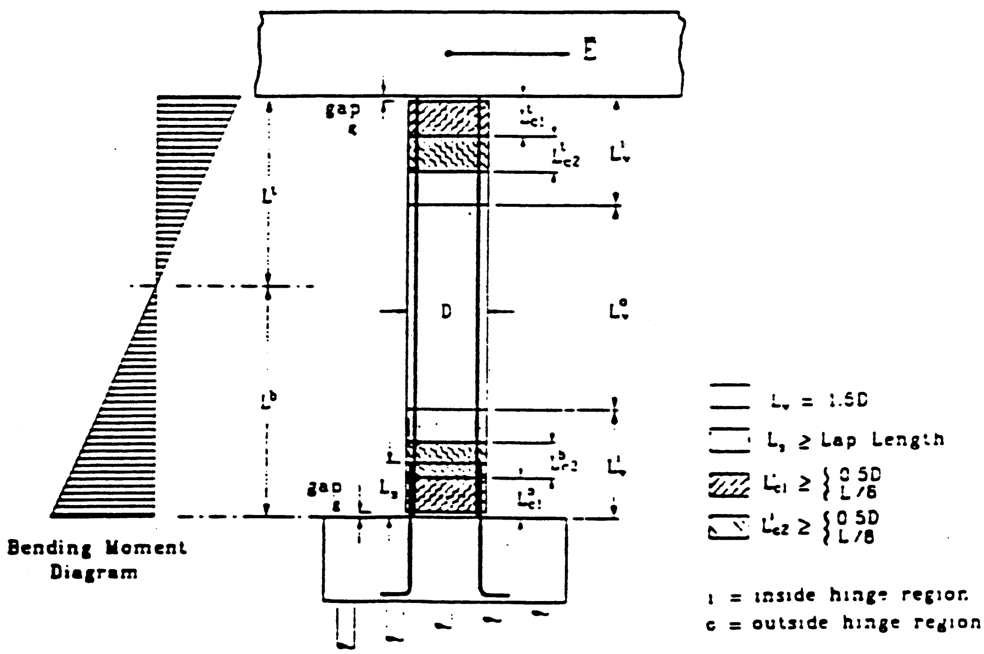


FIG 6. Carbon Jacket Regions for Bridge Column Retrofit. Double Bending

List of Reference Materials

1. Frieder Seible, M.J. Nigel Priestley, Donato Innamorato, *Earthquake Retrofit of Bridge column with Continuous Carbon Fiber Jackets*, Report No. ACTT-95/08, University of California at San Diego, August 1996

2. M.J.N. Priestley, F. Seible, G.M. Calvi, *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, Inc., 1996

BERGER/ABAM
ENGINEERS INC.

PLANNING
ENGINEERING
ENVIRONMENTAL
PROGRAM MANAGEMENT

Facsimile Cover Sheet

Fax ID Number 99 5/31

TO		FROM	
Name	David McLean	Name	Lee Marsh
Firm	WSU-CEE	Firm	*BERGER/ABAM* Engineers Inc.
Address	Pullman, WA	Address	33301 Ninth Avenue South Federal Way, WA 98003
Fax Number	509-335-7632	Fax Number	206/431-2250
cc:	Michael W LaNier	Telephone	206/431-2300

Total Number of Pages Transmitted *(Including Cover Sheet)* 7

Date Transmitted 5-13-99 **Time Transmitted** 2/0

Transmitted by Patty H. **BERGER/ABAM Job Number** P98108

RE: WSU / WSDOT Split Column Tests

David,

In a recent conversation with one of your researchers, Paul Rogness, it came to my attention that the calculations and design drawings for the transverse loading specimen of the above referenced tests did not properly include the splice confinement. The methodology that we followed, that developed by the University of California at San Diego, explicitly calculates the confinement for reinforcement lap splice zones. However, this methodology also recognizes that confinement of rectangular splice zones with rectangular jackets is not entirely possible. Since your tests were conducted on such a configuration, and since this effect was one of the parameters that you were investigating, I calculated the numbers of Replark 30 wraps that would have been recommended for the tests of the splice zone. The attached calculations indicate that 11 or so wraps would have been required. We understand that the performance of the specimens with the 6 wraps that were provided was fairly good. We would expect that if 11 wraps had been used, the performance may have been somewhat better. Therefore, the results probably form a lower bound to the performance expected if the calculated number of wraps are used. I hope that this additional information can be of help to you. If you have any questions, please give me a call.

Regards,

Lee

Lee

BERGER/ABAM Engineers Inc., 33301 Ninth Avenue South • Federal Way, WA 98003-6395
Phone 206/431-2300 • Fax 206/431-2250

BERGER/ABAM ENGINEERS INC

Project: Sumitomo Wrap
WSU Split Column Tests
Subject: Splice - Zone Confinement

Sheet 1 of 6
Job Number: F98100
Designer: LM
Date: 5/13/99

Jacket Thickness for equiv. circular column

(~~treat rectangular as equiv. circular col.~~)

$$t_j = 500 \frac{D(f_\ell - f_h)}{E_j}$$

$D \sim 16.33''$ equiv. dia. from prev. calc.

$f_\ell \approx 300$ psi *

f_h assume $= 0$

$E_j = 33425$ ksi

$$t_j \sim \frac{500(16.33'')(300 \text{ psi})}{33425 \text{ ksi} (1000) \text{ psi/ksi}} = \underline{\underline{0.0733''}}$$

Ref: ACTT-95/08 Earthquake Retrofit of Bridge Columns w/ Continuous Carbon Fiber Jackets p.4

$$\# \text{ wraps} = \frac{0.0733}{0.0066} \approx 11$$

Report so 11 wraps req'd

* Calc. f_ℓ: $$f_\ell = \frac{A_s f_y}{\left[\frac{p}{2n} + 2(d_b + cc)\right] L_s}$$

p = inside crack perimeter
d_b = bar dia.
n = # of bars
cc = clr cover
L_s = splice length

approx. $p \sim 16.33 - 0.67(2)\pi = 15''$

$n = 10$ bars

$d_b = 0.375''$

$cc = 0.67''$

$L_s = 7.5''$ $(20 d_b)$

$A_s = 0.11(10) = 1.1 \text{ in}^2$

$f_y = 44$ ksi

$$f_\ell = \frac{44(1.1)}{\left[\frac{15}{2(10)} + 2(0.375 + 0.67)\right] 7.5} = 0.264 \text{ ksi}$$

use 300 psi OK

ACTT-95/08 Does not recommend using rectangular jackets on rectangular columns to confine splice zones. If some debonding is permissible in the splice zone, then rect. jackets may be effective. Also typically the calc. jacket thickness would be doubled, producing 0.14 in. or 22 wraps.

001-001006

BERGER/ABAM ENGINEERS INC

Project: Sumitomo Wrap
WSU Split Column Tests
Subject: Splice Zone
Confinement

Sheet 2 of 6
Job Number: F96105
Designer: LH
Date: 5/13/99

Alternate method of determining jacket thickness

Ref: Seismic Design & Retrofit of Bridges
Priestley, Seible, Calvi 1996 §8.2.2(b)(iii)

$$\rho_{sj} = 2\frac{\left(A_b f_s / 4 p l_s - \cancel{f_3}^{\,0}\right)}{0.0015 E_j}$$

f_l

ACTT based on 0.001 E so this is less conservative

$$f_l = \frac{A_b f_s}{4 p l_s} \rightarrow \frac{1.21 A_b f_{yl}}{p l_s}$$

$$f_l = \frac{1.21(1.1)40}{15(7.5)} = 0.47 \text{ ksi}$$

$$\rho_{sj} = \frac{2(0.47)}{0.0015(33425)} = 0.0189$$

$$\rho_s = 2 t_j \left[\frac{b+h}{bh}\right]$$

for rectangle

7"

13"

$$t_j = \frac{\rho_s}{2}\frac{bh}{[b+h]} = \frac{0.0189}{2}\frac{(7)13''}{(7+13)''}$$

$$t_j = 0.043 \text{ in}$$

$$\#\text{ wraps} = \frac{0.043}{0.0066} = 6.5$$

Replace 30 wraps reqd

Note, if use $f_l = 0.47$ & $\epsilon = 0.0015$ in ACTT eqn

$$t_j = \frac{500(16.33).47}{33425}\left(\frac{.001}{.0015}\right) = 0.077$$

$$\#\text{ wraps} = \frac{0.077}{0.0066} = 11.6 \text{ wraps}$$

- or about the same as before.

but ACTT method may be a better predictor since ρ_s as calc. for a rectangle is not truly applicable.

APPENDIX C

XXSYS RETROFIT DESIGN CALCULATIONS

XXSYS "CHOPSTICK DESIGN"

Column Specifications:

Column Height,	$L := 60$	(in)
Shear Span,	$L_e := 30$	(in)
Column Width,	$B := 13$	(in)
Column Depth,	$D := 7$	(in)
Concrete Cover,	$cc := 0.67$	(in)
Concrete Compressive Strength,	$f_c := 5000$	(psi)

Longitudinal Reinforcement 10 #3	$f_{yl} := 40$	(ksi)
Number of Bars	$n := 10$	
Bar Diameter	$d_b := 0.375$	(in)
Bar Area	$A_b := 0.11$	(in^2)
Steel Modulus	$E_s := 29000$	(ksi)

Transverse Reinforcement 0.12 @ 4 in.	$f_{yt} := 40$	(ksi)
Bar Diameter	$d_s := 0.12$	(in)
Bar Area	$A_s := 0.011$	(in^2)
Spacing	$s := 4$	(in)

Lap Splice Length, $L_s := 7.5$ (in)

Section Properties:

Axial Load,	$P := 0$	(kips)
Moment Capacity,	$M_{yi} := 152.4$	(kip in)
Yield Curvature,	$\phi_y := 0.000426$	(1/in)
Neutral Axis Depth,	$c_u := 0.7$	(in)

Jacket Material Properties:

Jacket Modulus,	$E_j := 12000$	(ksi)
Ultimate Jacket Strength,	$f_{ju} := 120$	(ksi)
Ultimate Strain,	$\varepsilon_{ju} := 0.01$	(in/in)

Required Displacement Ductility: $\mu_\Delta := 8$

Shear Strength Retrofit:

Plastic shear including overstrength =

$$V_o := 1.5 \cdot \left(\frac{M_{yi}}{L_e} \right) \qquad V_o = 7.6 \quad \text{(kips)}$$

Concrete shear contribution =

$$k := \text{if}\left(\mu_\Delta < 2, 3, \text{if}\left(\mu_\Delta < 4, 5 - \mu_\Delta, \text{if}\left(\mu_\Delta < 8, 1.5 - \frac{\mu_\Delta}{8}, \text{if}\left(\mu_\Delta \geq 8, 0.5, 0.5\right)\right)\right)\right) \qquad k = 0.5$$

$$V_{ci} := k \cdot \sqrt{f_c} \cdot 0.8 \cdot (D \cdot B) \cdot 10^{-3} \qquad V_{ci} = 2.6 \quad \text{(kips)}$$

$$V_{co} := 3 \cdot \sqrt{f_c} \cdot 0.8 \cdot (D \cdot B) \cdot 10^{-3} \qquad V_{co} = 15.4 \quad \text{(kips)}$$

Hoop reinforcement shear contribution =

$$D_c := D - 2 \cdot cc + d_b$$

$$\theta := 45$$

$$\alpha := \theta \cdot \frac{\pi}{180} \quad \text{(change to radians)}$$

$$nbar := 2 \quad \text{(\# transverse bars)}$$

$$V_s := \frac{nbar \cdot A_s \cdot f_{yt} \cdot D_c}{s} \cdot \cot(\alpha) \qquad V_s = 1.3 \quad \text{(kips)}$$

Axial load shear contribution = $V_p := P \cdot \frac{D - c_u}{2 \cdot L_e}$ $\qquad V_p = 0 \quad$ (kips)

Jacket thickness inside the plastic hinge region t_{ji} =

Strength reduction factor ϕ $\quad \phi := 0.85$

$$t_{vi} := \frac{125}{(E_j \cdot D)} \cdot \left[\frac{V_o}{\phi} - \left(V_{ci} + V_s + V_p\right) \right] \qquad t_{vi} = 0.008 \quad \text{(in)} \quad <<<<<<$$

Jacket thickness outside the plastic hinge region t_{jo} =

$$t_{vo} := \frac{125}{(E_j \cdot D)} \cdot \left[\frac{V_o}{\phi} - \left(V_{co} + V_s + V_p\right) \right] \qquad t_{vo} = -0.012 \quad \text{(in)} \quad <<<<<<$$

Flexural Plastic Hinge Confinement:

Equivalent column diameter D_e =

$$A := D$$

$$k := \left(\frac{A}{B}\right)^{\frac{2}{3}}$$

$$b := \sqrt{\left[\frac{A}{(2\cdot k)}\right]^2 + \left(\frac{B}{2}\right)^2}$$

$$a := k\cdot b$$

$$R_1 := \frac{b^2}{a}$$

$$R_2 := \frac{a^2}{b}$$

$$D_e := R_1 + R_2 \qquad D_e = 16.331 \quad \text{(in)}$$

Plastic hinge length = $L_p := L_e\cdot 0.08 + 0.15\cdot f_{yl}\cdot d_b$ $\qquad L_p = 4.7$ (in)

Curvature ductility demand =

$$\mu_\phi := 1 + \frac{\mu_\Delta - 1}{3\cdot\left(\frac{L_p}{L_e}\right)\cdot\left[1 - 0.5\cdot\left(\frac{L_p}{L_e}\right)\right]} \qquad \mu_\phi = 17$$

Required ultimate compression strain in the concrete =

$$\varepsilon_{cu} := \mu_\phi\cdot\phi_y\cdot c_u \qquad \varepsilon_{cu} = 0.0052$$

Jacket thickness t_{c1} and t_{c2} =

$$t_{c1} := \left[0.1\cdot\frac{D_e\cdot\left(\varepsilon_{cu} - 0.004\right)\cdot 1.5\cdot f_c}{f_{ju}\cdot\varepsilon_{ju}}\cdot 10^{-3}\right]\cdot 2 \qquad t_{c1} = 0.024 \quad \text{(in)} \quad <<<<<<$$

$$t_{c2} := \frac{t_{c1}}{2} \qquad t_{c2} = 0.012 \quad \text{(in)} \quad <<<<<<$$

Thickness to prevent bar buckling = $t_b := \frac{n\cdot D'}{E_j}$ $\qquad t_b = 0.006$ (in)

Jacket Specifications:

Shear Strength Component =

$L_{vi} := 1.5 \cdot D$ $\qquad L_{vi} = 10.5$ (in) >>>>> $t_{vi} = 0.008$ (in)

$L_{vo} := L - 2 \cdot L_{vi}$ $\qquad L_{vo} = 39$ (in) >>>>> $t_{vo} = -0.012$ (in)

Confinement Component =

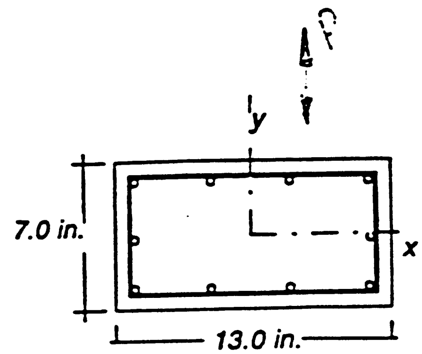
$L_{c1} := \mathrm{if}\left(0.5 \cdot D \geq 0.125 \cdot L_e, 0.5 \cdot D, 0.125 \cdot L_e\right)$ $\qquad L_{c1} = 3.75$ (in) >>>>> $t_{c1} = 0.024$ (in)

$L_{c2} := \mathrm{if}\left(0.5 \cdot D \geq 0.125 \cdot L_e, 0.5 \cdot D, 0.125 \cdot L_e\right)$ $\qquad L_{c2} = 3.75$ (in) >>>>> $t_{c2} = 0.012$ (in)

$t_b = 0.006$ (in)

SECTION PROPERTIES:

Section Depth	= 7.0 in.
Section Width	= 13.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Model	= Mander (no tensile strength)
Concrete Strength	= 5.00 ksi
Steel Model	= Mild Strength Steel
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Tension Side Reinforcement	= 4 Bars (#3 Bars)
Compression Side Reinforcement	= 4 Bars (#3 Bars)
Side Reinforcement	= 1 Bars (#3 Bars) each side



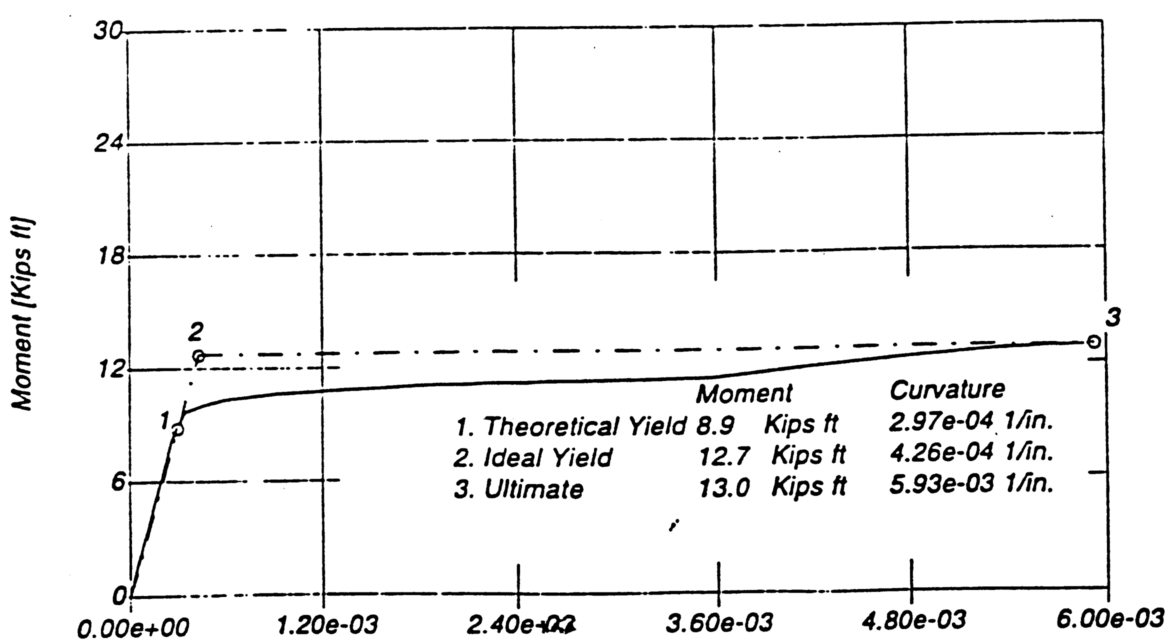
Hoop Size	= 0.12 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 4.0 in.	*Hoop Strength*	= 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:

The Ideal Moment Capacity is based on the concrete strain of 0.004.

Applied Axial Load	= 0.0 Kips	*Ag*	= 91.0 sq.in
EIeff	= 2.49e+03 Kips sq.ft	*Ig*	= 3.72e+02 in^4
Curvature Ductility	= 13.9	*Ieff*	= 8.43e+01 in^4

Moment Curvature Plot



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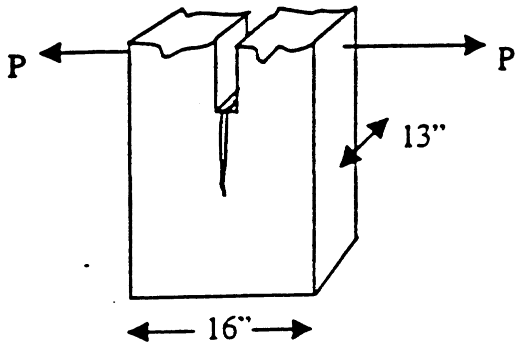
Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	1.5 in.	-0.00030	2.1 Kips ft	6.55e-05 1/in.	
0.0002	1.5 in.	-0.00060	4.1 Kips ft	1.31e-04 1/in.	
0.0003	1.5 in.	-0.00091	6.2 Kips ft	1.96e-04 1/in.	
0.0004	1.5 in.	-0.00121	8.3 Kips ft	2.62e-04 1/in.	
0.0005	1.5 in.	-0.00161	9.7 Kips ft	3.44e-04 1/in.	__Yield
0.0006	1.3 in.	-0.00224	10.1 Kips ft	4.63e-04 1/in.	
0.0007	1.2 in.	-0.00297	10.3 Kips ft	5.97e-04 1/in.	
0.0008	1.1 in.	-0.00377	10.5 Kips ft	7.44e-04 1/in.	
0.0009	1.0 in.	-0.00463	10.6 Kips ft	9.00e-04 1/in.	
0.0010	0.9 in.	-0.00553	10.7 Kips ft	1.06e-03 1/in.	
0.0011	0.9 in.	-0.00646	10.8 Kips ft	1.23e-03 1/in.	
0.0012	0.9 in.	-0.00741	10.9 Kips ft	1.40e-03 1/in.	
0.0013	0.8 in.	-0.00838	11.0 Kips ft	1.58e-03 1/in.	
0.0014	0.8 in.	-0.00935	11.0 Kips ft	1.75e-03 1/in.	
0.0015	0.8 in.	-0.01032	11.1 Kips ft	1.92e-03 1/in.	
0.0016	0.8 in.	-0.01128	11.1 Kips ft	2.10e-03 1/in.	
0.0017	0.7 in.	-0.01224	11.2 Kips ft	2.27e-03 1/in.	
0.0018	0.7 in.	-0.01319	11.2 Kips ft	2.44e-03 1/in.	
0.0019	0.7 in.	-0.01412	11.2 Kips ft	2.61e-03 1/in.	
0.0020	0.7 in.	-0.01503	11.2 Kips ft	2.77e-03 1/in.	
0.0025	0.7 in.	-0.01936	11.3 Kips ft	3.56e-03 1/in.	
0.0030	0.7 in.	-0.02290	12.0 Kips ft	4.22e-03 1/in.	
0.0035	0.7 in.	-0.02613	12.4 Kips ft	4.82e-03 1/in.	
0.0040	0.7 in.	-0.02913	12.7 Kips ft	5.39e-03 1/in.	
0.0045	0.8 in.	-0.03192	13.0 Kips ft	5.93e-03 1/in.	

Ultimate concrete strain was exceeded.

"Chopsticks" Design – Part 2



Goal: limit crack width to 1 mm in the 16" x 13" wide section to assure aggregate interlock in the concrete.

1) 1 mm x 1 in/25.4 mm = .03937" total crack width in concrete section

2) This crack width spread over the 16" wide section would be:
.03937"/16" = .00246 in/in of width = .246% strain = the strain limit in composite

3) Now, using: $\sigma_{applied} = \varepsilon E$
then: $\sigma_{applied} = (.00246)(12{,}000{,}000)$
$= 29{,}520$ lbs/in^2 would be developed in the composite at that strain level.

4) Using a material with a strength of 120,000 lb/in^2, we would need 29,520/120,000 or .246 in^2 of material to restrain the concrete.

5) If this area was spread out over 6", it would be .041" thick (.246in^2/6" = .041")
Over 12", it would be .0205" thick.

6) Therefore, as a safe design that both restrains the crack and allows for a smooth transition of load resistance, we will apply (2) layers of .0266" thick GA180 fabric to the column: (1) 12" wide section overlaid by a (1) 6"-wide section applied to the top 6". The bottom 6" would have a thickness of .0266" and the top 6" would have a thickness of .0532".

XXSYS "GROUTED" COLUMN DESIGN

Column Specifications:

Column Height,	$L := 104$	(in)
Shear Span,	$L_e := 104$	(in)
Column Width,	$B := 16$	(in)
Column Depth,	$D := 13$	(in)
Concrete Cover,	$cc := 0.67$	(in)
Concrete Compressive Strength,	$f_c := 5000$	(psi)

Longitudinal Reinforcement 20 #3	$f_{yl} := 40$	(ksi)
Number of Bars	$n := 16$	
Bar Diameter	$d_b := 0.375$	(in)
Bar Area	$A_b := 0.11$	(in^2)
Steel Modulus	$E_s := 29000$	(ksi)

Transverse Reinforcement 0.12 @ 4 in.	$f_{yt} := 40$	(ksi)
Bar Diameter	$d_s := 0.12$	(in)
Bar Area	$A_s := 0.011$	(in^2)
Spacing	$s := 4$	(in)

Lap Splice Length, $L_s := 7.5$ (in)

Section Properties:

Axial Load,	$P := 0$	(kips)
Moment Capacity,	$M_{yi} := 547.2$	(kip in)
Yield Curvature,	$\phi_y := 0.000235$	(1/in)
Neutral Axis Depth,	$c_u := 0.9$	(in)

Jacket Material Properties:

Jacket Modulus,	$E_j := 12000$	(ksi)
Ultimate Jacket Strength,	$f_{ju} := 120$	(ksi)
Ultimate Strain,	$\varepsilon_{ju} := 0.01$	(in/in)

Required Displacement Ductility: $\mu_\Delta := 8$

Shear Strength Retrofit:

Plastic shear including overstrength =

$$V_o := 1.5\cdot\left(\frac{M_{yi}}{L_e}\right) \qquad V_o = 7.9 \quad \text{(kips)}$$

Concrete shear contribution =

$$k := \text{if}\left(\mu_\Delta<2,3,\text{if}\left(\mu_\Delta<4,5-\mu_\Delta,\text{if}\left(\mu_\Delta<8,1.5-\frac{\mu_\Delta}{8},\text{if}\left(\mu_\Delta\geq 8,0.5,0.5\right)\right)\right)\right) \qquad k = 0.5$$

$$V_{ci} := k\cdot\sqrt{f_c}\cdot 0.8\cdot(D\cdot B)\cdot 10^{-3} \qquad V_{ci} = 5.9 \quad \text{(kips)}$$

$$V_{co} := 3\cdot\sqrt{f_c}\cdot 0.8\cdot(D\cdot B)\cdot 10^{-3} \qquad V_{co} = 35.3 \quad \text{(kips)}$$

Hoop reinforcement shear contribution =

$$D_c := D - 2\cdot cc + d_b$$

$$\theta := 45$$

$$\alpha := \theta\cdot\frac{\pi}{180} \quad \text{(change to radians)}$$

$$\text{nbar} := 2 \quad \text{(\# transverse bars)}$$

$$V_s := \frac{\text{nbar}\cdot A_s\cdot f_{yt}\cdot D_c}{s}\cdot\cot(\alpha) \qquad V_s = 2.6 \quad \text{(kips)}$$

Axial load shear contribution = $V_p := P\cdot\dfrac{D - c_u}{2\cdot L_e}$ $\qquad V_p = 0 \quad$ (kips)

Jacket thickness inside the plastic hinge region t_{ji} =

Strength reduction factor ϕ $\quad \phi := 0.85$

$$t_{vi} := \frac{125}{(E_j\cdot D)}\cdot\left[\frac{V_o}{\phi} - (V_{ci} + V_s + V_p)\right] \qquad t_{vi} = 6.044\cdot 10^{-4}\,\text{(in)} \quad \text{<<<<<<}$$

Jacket thickness outside the plastic hinge region t_{jo} =

$$t_{vo} := \frac{125}{(E_j\cdot D)}\cdot\left[\frac{V_o}{\phi} - (V_{co} + V_s + V_p)\right] \qquad t_{vo} = -0.023 \quad \text{(in)} \quad \text{<<<<<<}$$

Flexural Plastic Hinge Confinement:

Equivalent column diameter D_e =

$$A := D$$

$$k := \left(\frac{A}{B}\right)^{\frac{2}{3}}$$

$$b := \sqrt{\left[\frac{A}{(2\cdot k)}\right]^2 + \left(\frac{B}{2}\right)^2}$$

$$a := k\cdot b$$

$$R_1 := \frac{b^2}{a}$$

$$R_2 := \frac{a^2}{b}$$

$$D_e := R_1 + R_2 \qquad D_e = 20.862 \quad \text{(in)}$$

Plastic hinge length = $L_p := L_e\cdot 0.08 + 0.15\cdot f_{yl}\cdot d_b$ $\quad L_p = 10.6 \quad$ (in)

Curvature ductility demand =

$$\mu_\phi := 1 + \frac{\mu_\Delta - 1}{3\cdot\left(\frac{L_p}{L_e}\right)\cdot\left[1 - 0.5\cdot\left(\frac{L_p}{L_e}\right)\right]} \qquad \mu_\phi = 25$$

Required ultimate compression strain in the concrete =

$$\varepsilon_{cu} := \mu_\phi\cdot\phi_y\cdot c_u \qquad \varepsilon_{cu} = 0.0053$$

Jacket thickness t_{c1} and t_{c2} =

$$t_{c1} := \left[0.1\cdot\frac{D_e\cdot\left(\varepsilon_{cu} - 0.004\right)\cdot 1.5\cdot f_c}{f_{ju}\cdot\varepsilon_{ju}}\cdot 10^{-3}\right]\cdot 2 \qquad t_{c1} = 0.035 \quad \text{(in)} \quad <<<<<<$$

$$t_{c2} := \frac{t_{c1}}{2} \qquad t_{c2} = 0.017 \quad \text{(in)} \quad <<<<<<$$

Thickness to prevent bar buckling = $t_b := \frac{n\cdot D'}{E_j}$ $\quad t_b = 0.017 \quad$ (in)

Lap Splice Clamping:

Available lateral clamping pressure =

$$f_h := 0 \qquad f_h = 0 \quad \text{(ksi)}$$

Required clamping pressure =

$$dd := \left[D - 2\cdot\left(d_b + cc\right)\right]$$

$$bb := \left[B - 2\cdot\left(d_b + cc\right)\right]$$

$$p := 2\cdot(dd + bb)$$

$$f_l := \frac{A_b \cdot f_{yl}}{\left[\dfrac{p}{2\cdot n} + 2\cdot\left(d_b + cc\right)\right]\cdot L_s} \qquad f_l = 0.161 \quad \text{(ksi)}$$

Required jacket thickness =

$$t_s := \left[500\cdot\left(\frac{D_e}{E_j}\right)\cdot\left(f_l - f_h\right)\right]\cdot 2 \qquad t_s = 0.28 \quad \text{(in)}$$

<<<<<<

Jacket Specifications:

Shear Strength Component =

$L_{vi} := 1.5 \cdot D$ $\qquad L_{vi} = 19.5$ (in) >>>>> $t_{vi} = 6.044 \cdot 10^{-4}$ (in)

$L_{vo} := L - 2 \cdot L_{vi}$ $\qquad L_{vo} = 65$ (in) >>>>> $t_{vo} = -0.023$ (in)

Confinement Component =

$L_{c1} := \text{if}\left(0.5 \cdot D \geq 0.125 \cdot L_e, 0.5 \cdot D, 0.125 \cdot L_e\right)$ $\qquad L_{c1} = 13$ (in) >>>>> $t_{c1} = 0.035$ (in)

$L_{c2} := \text{if}\left(0.5 \cdot D \geq 0.125 \cdot L_e, 0.5 \cdot D, 0.125 \cdot L_e\right)$ $\qquad L_{c2} = 13$ (in) >>>>> $t_{c2} = 0.017$ (in)

$t_b = 0.017$ (in)

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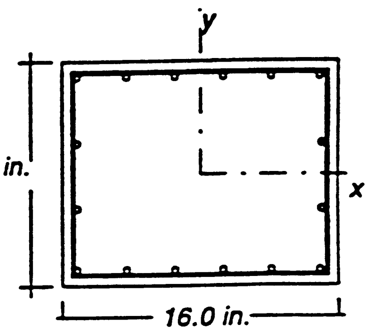
SECTION PROPERTIES:

Section Depth	= 13.0 in.
Section Width	= 16.0 in.
Cover to Main Steel	= 0.7 in.
Concrete Model	= Mander (no tensile strength)
Concrete Strength	= 5.00 ksi
Steel Model	= Mild Strength Steel
Steel Strength	= 40.0 ksi
Young's Modulus	= 29000.0 ksi
Tension Side Reinforcement	= 6 Bars (#3 Bars)
Compression Side Reinforcement	= 6 Bars (#3 Bars)
Side Reinforcement	= 2 Bars (#3 Bars) each side

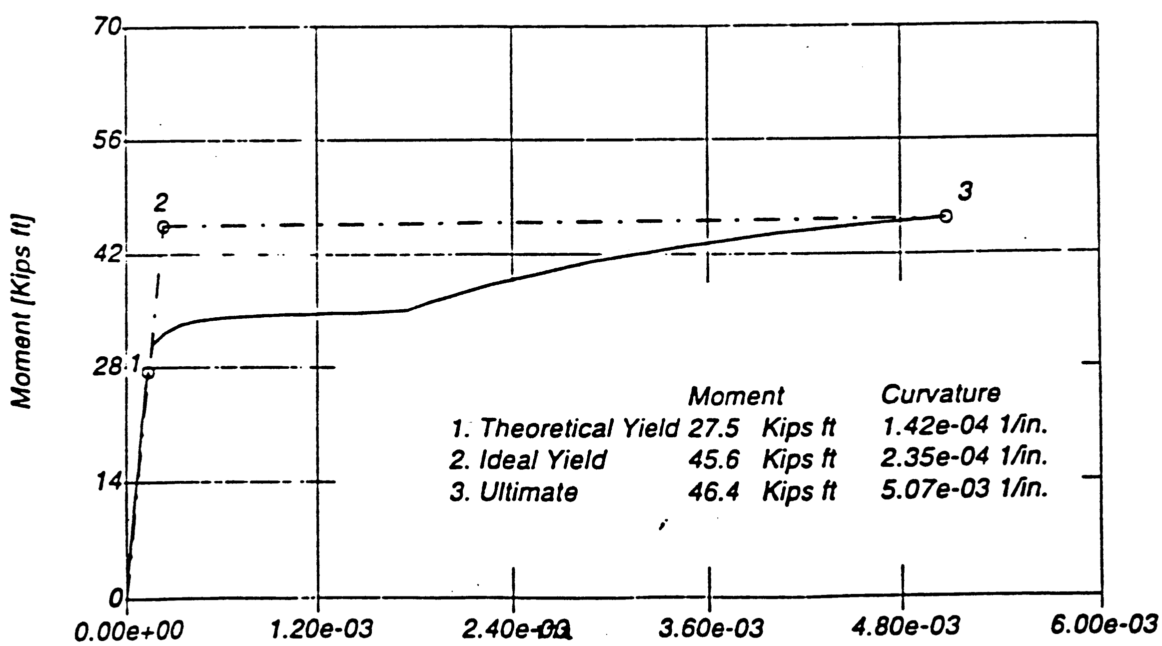


Hoop Size	= 0.12 Bars	*Average Number of Legs*	= 2.0
Hoop Spacing	= 4.0 in.	*Hoop Strength*	= 40.0 ksi

MOMENT CURVATURE ANALYSIS RESULTS:
The Ideal Moment Capacity is based on the concrete strain of 0.004.

Applied Axial Load	= 0.0 Kips	*Ag*	= 208.0 sq.in
EIeff	= 1.62e+04 Kips sq.ft	*Ig*	= 2.93e+03 in^4
Curvature Ductility	= 21.6	*Ieff*	= 5.48e+02 in^4

Moment Curvature Plot



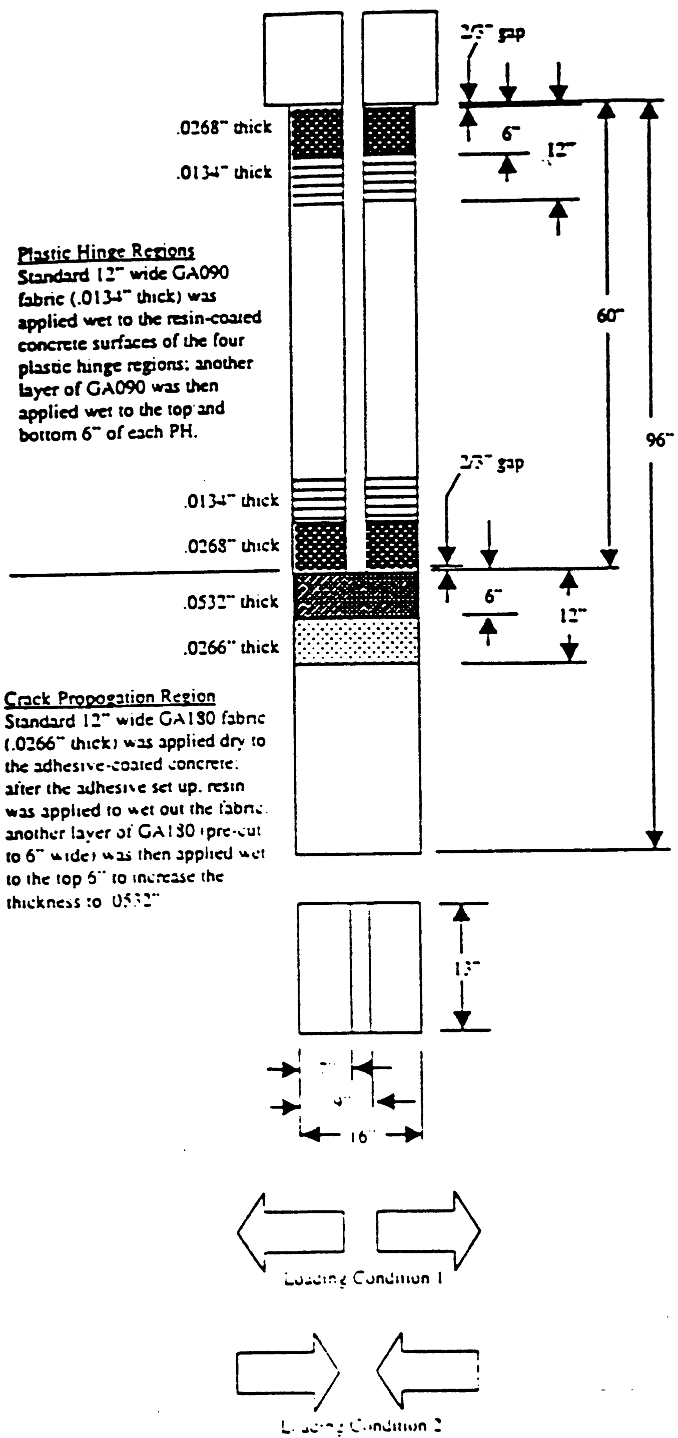
EVALUATION RELEASE OF SEQMC MAY NOT BE UTILIZED FOR PRODUCTION.

Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	2.5 in.	-0.00039	8.4 Kips ft	4.02e-05 1/in.	
0.0002	2.5 in.	-0.00078	16.8 Kips ft	8.05e-05 1/in.	
0.0003	2.5 in.	-0.00116	25.2 Kips ft	1.21e-04 1/in.	
0.0004	2.3 in.	-0.00170	31.0 Kips ft	1.73e-04 1/in.	__Yield
0.0005	2.0 in.	-0.00256	32.5 Kips ft	2.52e-04 1/in.	
0.0006	1.7 in.	-0.00360	33.3 Kips ft	3.46e-04 1/in.	
0.0007	1.5 in.	-0.00482	33.8 Kips ft	4.55e-04 1/in.	
0.0008	1.4 in.	-0.00618	34.1 Kips ft	5.75e-04 1/in.	
0.0009	1.3 in.	-0.00767	34.3 Kips ft	7.06e-04 1/in.	
0.0010	1.2 in.	-0.00926	34.5 Kips ft	8.45e-04 1/in.	
0.0011	1.1 in.	-0.01093	34.6 Kips ft	9.91e-04 1/in.	
0.0012	1.1 in.	-0.01264	34.7 Kips ft	1.14e-03 1/in.	
0.0013	1.0 in.	-0.01440	34.8 Kips ft	1.29e-03 1/in.	
0.0014	1.0 in.	-0.01619	34.9 Kips ft	1.45e-03 1/in.	
0.0015	0.9 in.	-0.01798	35.0 Kips ft	1.60e-03 1/in.	
0.0016	0.9 in.	-0.01968	35.2 Kips ft	1.75e-03 1/in.	
0.0017	0.9 in.	-0.02128	36.2 Kips ft	1.89e-03 1/in.	
0.0018	0.9 in.	-0.02275	36.9 Kips ft	2.02e-03 1/in.	
0.0019	0.9 in.	-0.02422	37.7 Kips ft	2.15e-03 1/in.	
0.0020	0.9 in.	-0.02567	38.4 Kips ft	2.28e-03 1/in.	
0.0025	0.9 in.	-0.03266	41.2 Kips ft	2.90e-03 1/in.	
0.0030	0.9 in.	-0.03927	43.2 Kips ft	3.48e-03 1/in.	
0.0035	0.9 in.	-0.04548	44.6 Kips ft	4.03e-03 1/in.	
0.0040	0.9 in.	-0.05141	45.6 Kips ft	4.56e-03 1/in.	
0.0045	0.9 in.	-0.05708	46.4 Kips ft	5.07e-03 1/in.	

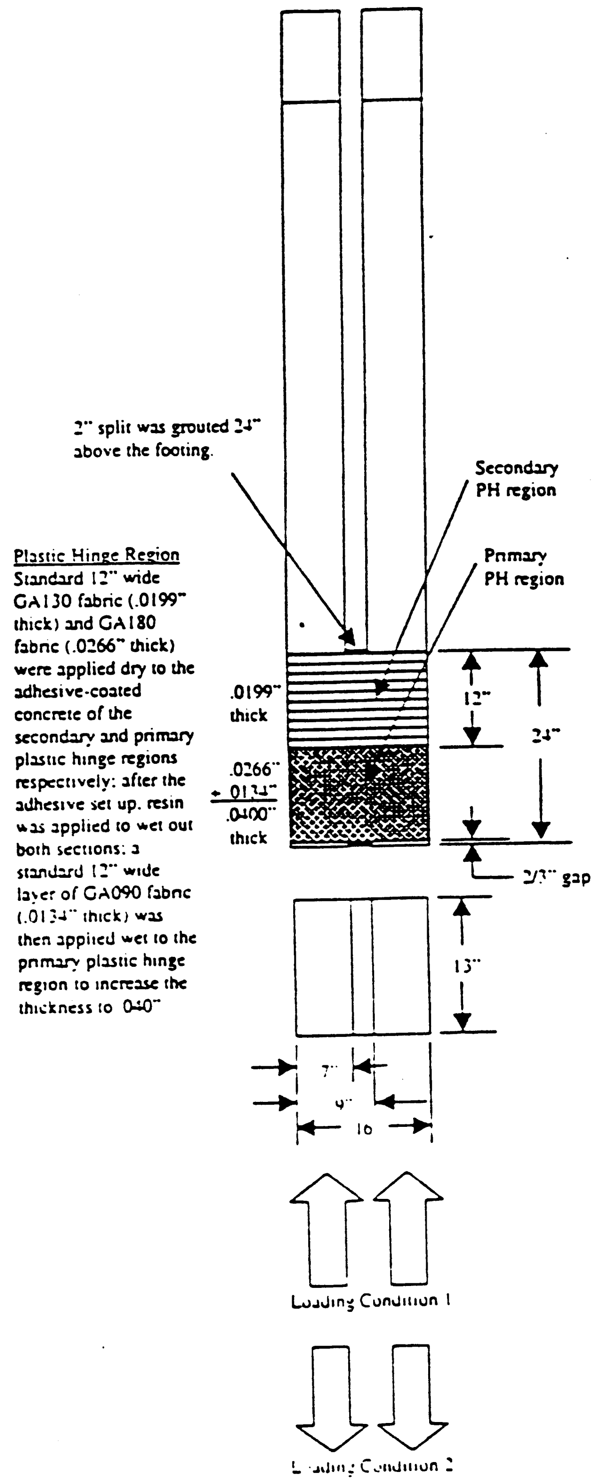
Ultimate concrete strain was exceeded.

Composite Lay-up for WSU Split Column Tests

Out-of-Phase, Longitudinal Loading, Double Bending
("Chopsticks")



Transverse Loading, Single Bending
("Grouted")



Quantities of products used to construct the lay-ups

Carbon Fiber Fabric	"Chopsticks" Plastic Hinges (4 places)	"Chopsticks" Crack	"Grouted" Plastic Hinge
GA090 8.9 oz/yd² .0134" thick	(1) 47¼ x 12" (1) 47¼" x 6"	–	(1) 65" x 12" (lower 12")
GA130 13.2 oz/yd² .0199" thick	–	–	(1) 65" x 12" (upper 12")
GA180 17.7 oz/yd² .0266" thick	–	(1) 65" x 12" (1) 65" x 6"	(1) 65" x 12" (lower 12")

(all fabrics are 12K, all come 12" wide standard (6" wide sections had to be cut), thickness assumes 50% V_f)

Adhesive	"Chopsticks" Plastic Hinges (4 places)	"Chopsticks" Crack	"Grouted" Plastic Hinge
Bondtite R37	not used	1.210 lbs.	2.37 lbs.
Bondtite H37	not used	.403 lbs.	.79 lbs.

(3:1 mix ratio; coverage rate: .333lbs/ft²)

Resin	"Chopsticks" Plastic Hinges (4 places)	"Chopsticks" Crack	"Grouted" Plastic Hinge
826	(2) 2.274 lbs.	2.178 lbs.	3.18 lbs.
3379	(2) 1.170 lbs.	1.122 lbs.	1.62 lbs.

Note: 2 kits of the size shown were made to wet out the "Chopsticks" PH regions - 1 kit was made for the two "bottom" PHs and another kit was made for the two "top" PHs. Since the "top" PHs were the last to be applied and wetted out, an additional .379 lbs. of 826 and .195 lbs. of 3379 were added to the kit in order to have some resin left over to wet out any "dry" or "drier looking" areas on either test column. As it turned out, this extra resin wasn't really needed, but was applied anyway.