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Title/Desc:

305 BLDG 2 TON BRIDGE CRANE & MONORAIL ASSEMBLY
ANALYSIS

Pages: 56

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ENGINEERING DATA TRANSMITTAL

Page 1 of 1

1. EDT

No 614408

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(A) Item No.	(B) Document/Drawing No.	(C) Sheet No.	(D) Rev. No.	(E) Title or Description of Data Transmitted	Approval Designator	Reason for Transmittal	Originator Disposition	Receiver Disposition
1	WHC-SD-SNF-DA-016	N/A	0	305 Building 2 Ton Bridge Crane and Monorail Assembly Analysis	SQ	1	1	

16. KEY					
Approval Designator (F)		Reason for Transmittal (G)		Disposition (H) & (I)	
E, S, Q, D or N/A (see WHC-CM-3-5, Sec. 12.7)	1. Approval 2. Release 3. Information	4. Review 5. Post-Review 6. Dist. (Receipt Acknow. Required)	1. Approved 2. Approved w/comment 3. Disapproved w/comment	4. Reviewed no/comment 5. Reviewed w/comment 6. Receipt acknowledged	

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Reason	Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN	(J) Name	(K) Signature	(L) Date	(M) MSIN	Reason	Disp.
1	1	Cog. Eng. L. H. Goldmann	<i>[Signature]</i>	1/24/96	R3-86	T. E. Griffin, Engineer	<i>[Signature]</i>	1/23/96	6-13	1	1
1	1	Cog. Mgr. J. R. Frederickson	<i>[Signature]</i>	1/23/96	R3-86	M. J. Schliebe, Manager	<i>[Signature]</i>	1/23/96	6-13	1	1
1	1	QA M. S. Calhoun	<i>[Signature]</i>	1-30-96							
1	1	Safety SA K. Hartung	<i>[Signature]</i>	1-26-96	66-57						
		Env.									
1	1	Eng. Mgr. S. K. Farnworth	<i>[Signature]</i>	1/23/96	R3-86						
1	1	Indep. Review T. C. Mackey	<i>[Signature]</i>	1/24/96	S2-03						

18. M. D. Axup <i>[Signature]</i> 1-24-96 Signature of EDT Originator		19. L. H. Goldmann <i>[Signature]</i> 1/31/96 Authorized Representative Date for Receiving Organization		20. J. R. Frederickson <i>[Signature]</i> 1/31/96 Cognizant Manager Date		21. DOE APPROVAL (if required) Ctrl. No. ☐ Approved ☐ Approved w/comments ☐ Disapproved w/comments	
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305 BUILDING 2 TON BRIDGE CRANE AND MONORAIL ASSEMBLY ANALYSIS

M. D. Axup
ICF KH, Richland, WA 99352
U.S. Department of Energy Contract DE-AC06-87RL10930

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Key Words: Analysis, 305, Bridge Crane, Monorail

Abstract: The analyses in the appendix of this document evaluate the integrity of the existing bridge crane structure, as depicted on drawing H-3-34292, for a bridge crane and monorail assembly with a load rating of 2 tons. This bridge crane and monorail assembly is a modification of a 1 1/2 ton rated manipulator bridge crane which originally existed in the 305 building.

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305 BUILDING
2 TON BRIDGE CRANE AND MONORAIL ASSEMBLY ANALYSIS

December 1995

PREPARED BY: M. D. Axup 12/21/95
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ICF Kaiser Hanford Company
Hanford Operations and Engineering Contractor
for the
U. S. Department of Energy
Richland, Washington

CHECKLIST FOR INDEPENDENT REVIEW

Document Reviewed 305 BUILDING 2 TON BRIDGE CRANE AND MONORAIL ASSEMBLYAuthor M. D. Axup

<u>Yes</u>	<u>No</u>	<u>N/A</u>	
☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☐	☐	☒	Computer codes and data files documented.
☒	☐	☐	Data used in calculations explicitly stated in document.
☒	☐	☐	Data checked for consistency with original source information as applicable.
☒	☐	☐	Mathematical derivations checked including dimensional consistency of results.
☐	☐	☒	Models appropriate and used within range of validity or use outside range of established validity justified.
☒	☐	☐	Hand calculations checked for errors.
☐	☐	☒	Code run streams correct and consistent with analysis documentation.
☐	☐	☒	Code output consistent with input and with results reported in analysis documentation.
☒	☐	☐	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
☒	☐	☐	Safety margins consistent with good engineering practices.
☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.

 T. C. Mackey, Reviewer

Date

1/9/96

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1.0 INTRODUCTION

The analyses in the appendix of this document evaluate the structural integrity of the existing bridge crane structure, as depicted on drawing H-3-34292 (WHC 1977), for a bridge crane and monorail assembly with a load rating capacity of 2 tons. This bridge crane and monorail assembly is a modification of a 1 1/2 ton rated manipulator bridge crane which originally existed in the 305 building. See Engineering Change Notice (ECN) #628549 for a schematic depiction of modification referencing bridge crane load ratings and Certified Vendor Information (CVI) file number.

2.0 SUMMARY

The analyses in the appendix of this document show that the 305 building crane structure (WHC 1977) is adequate to support the 2 ton bridge crane and monorail assembly including the effects of impact (1.10 factor) as defined in the Manual of Steel Construction (AISC 1989). Loading for the bridge crane and monorail assembly may be any combination as long as the combined loading of either system is less than or equal to 2 tons. In addition, the analyses show that the crane structure is adequate for simultaneous loading of an existing 5 ton bridge crane, supported by the top crane rails, and loading of the new 2 ton bridge crane and monorail assembly, supported by the lower crane rails.

3.0 DISCUSSION

The modification of the original 1 1/2 ton manipulator bridge crane is required to provide a mockup of a K basin monorail. The mockup is to be used to test and practice the handling of spent fuel in the K basins. The original manipulator bridge crane was supplied by The Bartholomew Co., Inc. and they were also chosen to supply the modification per WHC 1995a in accordance with WHC 1995b.

4.0 RESULTS/CONCLUSIONS

The structural analysis show that the existing design of the 305 building bridge crane structure is adequate to support the new 2 ton bridge crane and monorail assembly.

5.0 REFERENCES

- AISC, 1989, Manual of Steel Construction, ninth Edition, American Institute of Steel Construction, Chicago, Illinois.
- WHC, 1977, Structural Plan, Elevations & Details, drawing H-3-34292, Westinghouse Hanford Company, Richland, Washington.
- WHC, 1995a, Purchase Order, MDK-XDW-430047, Westinghouse Hanford Company, Richland, Washington.
- WHC, 1995b, Specification For Modification of Manipulator Crane, WHC-S-047, Westinghouse Hanford Company, Richland, Washington.

Appendix A:

305 Building 2 Ton Bridge Crane and Monorail Assembly Structural Analysis

KAISER ENGINEERS HANFORD	CALCULATION IDENTIFICATION AND INDEX	Page 1 of 1 Date Dec. 11, 1995
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This sheet shows the status and description of the attached Design Analysis sheets.

Discipline Civil/Structural WO/Job No. **E50071/TTCN LE112** Calculation No. **n/a**
Project No. & Name **Spent Nuclear Fuels** **KPN 797300**
Calculation Item **305 Building 2 Ton Bridge Crane and Monorail Assembly Analysis**

These calculations apply to:

Dwg. No. **H-3-34292** Rev. No. **2**
Dwg. No. **n/a** Rev. No. **n/a**
Other (Study, CDR) **ECN 628549** Rev. No. **n/a**

The status of these calculations is:

☐ Preliminary Calculations
☒ Final Calculations
☐ Check Calculations (On Calculation Dated)
☐ Void Calculation (Reason Voided)

Incorporated in Final Drawings? **in ECN** ☒ Yes ☐ No
This calculation verified by independent "check" calculations? ☐ Yes ☒ No

Original and Revised Calculation Approvals:

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	<i>Mark D. Arup</i> 12/13/95		
Checked by	<i>P.C. Mackey</i> 12/21/95		
Approved by	<i>SK Janke</i> 1/2/96		
Checked Against Approved Vendor Data			

<u>INDEX</u>	
Design Analysis Page No.	Description
i	Calculation ID/ Index
1	I Objective, II Design Inputs: Criteria, Assumptions, Methods to Be Used, References, III Conclusion
2-15	Calculations
Attachment 1	Vendor Bridge Crane and Monorail Assembly Calculations Including Kaiser Alternate Check Calculations
Attachment 2	Letter for 305 Building Crane Structure Analysis by Gary Wagenblast

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HANFORD

Calc. No. N/A

Revision 0

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DESIGN ANALYSIS

Client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *J.C. Mackey*

Revised

By

I. OBJECTIVE

Provide a design analysis to qualify the existing crane structure in the 305 building, as depicted on drawing H-3-34292, for a 2 ton crane rating supported on the lower crane rails. This rating is for the modified bridge crane and monorail assembly. Modifications were supplied by The Bartholomew Co., Inc. as per specification WHC-S-047 and purchase order MDK-XDW-430047.

II. DESIGN INPUTS**A. CRITERIA**

1. Bridge crane and monorail assembly to be rated for a 2 ton capacity.
2. Steel shapes and plates are ASTM-A36.
3. Welding electrodes are E70XX.
4. ~~BUILDING SAFETY CLASSIFICATION IS S.C. 3~~

B. ASSUMPTIONS

None

C. METHODS TO BE USED

1. Hand calculations.

D. REFERENCES

1. AISC Manual of Steel Construction, Allowable Stress Design, Ninth Edition.
2. AISC Manual of Steel Construction, Allowable Stress Design, Seventh Edition.
3. Design of Welded Structures, by Omer Blodgett.
4. Westinghouse Specification WHC-S-047, Specification For Modification Of Manipulator Bridge Crane.
5. Westinghouse Purchase Order MDK-XDW-430047.
6. ~~STRUCTURAL DESIGN CRITERIA, SDC 41, REV. 12~~

E. CALCULATIONS

Refer to following pages for design calculations.

III. CONCLUSION

The structural analysis show that the existing crane structure, as depicted on drawing H-3-34292, is adequate to support the 2 ton bridge crane and monorail assembly. Loading for the bridge crane and monorail assembly may be any combination as long as the combined loading of either system is less than or equal to 2 tons. In addition, this calculation qualifies the existing crane structure for simultaneous loading of the 5 ton bridge crane, supported by the top crane rails, and loading of the new 2 ton bridge crane and monorail assembly, supported by the lower crane rails.

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HANFORD

Calc. No. N/A

Revision 0

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DESIGN ANALYSIS

Client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *J.C. Mackey*

Revised

By

Load Condition 1: Monorail loaded at maximum with load centered on exist'g. bridge girder. Monorail is positioned at end of bridge girder. Also, 2 ton hoist is unloaded, and positioned at end of bridge girder.

PLAN

NOTE: MONORAIL & TROLLEY ARE
 POSITIONED @ STOPS

DESIGN LOADING & MAIL WEIGHTS

(FOR SECTIONS REF., SEE ATTACHMENT 1)

1. LIFTED LOAD = 4000 #
2. HOIST & TROLLEY = 1145 # SECTION 1.0
3. ORIGINAL BRIDGE GIRDER
 END TRUCK = 478 # SECTION 3.0
4. MONORAIL HOIST
 & TROLLEY = 411 # SECTION 6.0
5. 1/2 MONORAIL = 111 #
6. 1/2 FESTOON = 30 #
7. COL. & BRACE = 288 #
8. MONORAIL END TRUCK = 323 #
- COMBINED 1163 #
- (ABOVE 5 ITEMS)
9. SLAVE GIRDER END TRUCK = 300 # ESTIMATED
10. USE 70 #/ft FOR W14x61 + W18x40.

CALCULATE MAX. WHEEL LOADS FOR POINTS A, B & C

POINT A

$$\text{MAX.} = \left[0.070(31.0) + 0.478 \right] \frac{1}{2} + \left[4.0(1.25) + 1.163 \right] \left(\frac{27.5}{31.0} \right) + 1.145 \left(\frac{1}{2} \right) \left(\frac{28.75}{31.0} \right) = 7.32 \text{ K} \rightarrow \text{USE}$$

TEST LOAD FACTOR
OR

$$\text{MAX.} = \left[\left[0.070(31.0) + 0.478 \right] \frac{1}{2} + \left[4.0 + 1.163 \right] \left(\frac{27.5}{31.0} \right) + 1.145 \left(\frac{1}{2} \right) \left(\frac{28.75}{31.0} \right) \right] 1.10 = 7.08 \text{ K}$$

IMPACT

POINT B

$$\text{MAX.} = \left[\left[0.070(31.0) + 0.478 \right] \frac{1}{2} + 1.145 \left(\frac{1}{2} \right) \left(\frac{28.75}{31.0} \right) \right] 1.10 = 2.04 \text{ K}$$

POINT C

$$\text{MAX.} = \left[0.070(31.0) \frac{1}{2} + 0.300 + \left[1.163 - 0.411 \right] \left(\frac{27.5}{31.0} \right) \right] 1.10 = 2.26 \text{ K}$$

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

Client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *C. Maskey*

Revised

By

Load Condition 1: (Cont'd.)CALCULATE C.G. OF WHEEL LOADS

$$C.G. = \frac{2.04(3.17) + 2.26(10.0)}{7.32 + 2.04 + 2.26} = 2.50'$$

By inspection, max. moment in crane rail will occur under the loaded W14x61 when the loads are positioned as shown below. Ref. AISC manual, p. 2-310, "General Rules for Simple Beams Carrying Moving Concentrated Loads"

Use moment distribution to calculate maximum bending moments in crane rail. Conservatively, treat crane rail as a two span continuous beam.

DISTANCE FROM E
 COLUMN TO POINT A = $[20.0 - 2.50] / 2 = 8.75'$

CALCULATE FIXED END MOMENTS @ POINTS 1, 2 & 3 - Ref. AISC MAN., p. 2-301
 LOAD CASE 15 AND 17

$$M_{1, FEM} = - \left[\frac{0.060(20.0)^2}{12} + \frac{7.32(8.75)(11.25)}{20.0^2} + \frac{2.04(11.92)(8.08)}{20.0^2} + \frac{2.26(18.75)(1.25)}{20.0^2} \right] = -26.40^{1-K}$$

$$M_{2, FEM} = \frac{0.060(20.0)^2}{12} + \frac{7.32(11.25)(4.75)}{20.0^2} + \frac{2.04(8.08)(11.92)}{20.0^2} + \frac{2.26(1.25)(18.75)}{20.0^2} = +26.10^{1-K}$$

$$M_{2, R} = - \frac{0.060(20.0)^2}{12} = -2.0^{1-K}$$

$$M_{3, FEM} = \frac{0.060(20.0)^2}{12} = +2.0^{1-K}$$

DESIGN ANALYSIS

Client Westinghouse
Subject Crane Rail Analysis for 2 Ton Monorail
and 2 Ton Bridge Crane
Location 305 Bldg.

WO/Job No. LE112/KPCN P97000

Date 12/05/95 By Mark Axup

Checked 12/21/95 By *C. Mackay*

Revised

By

Load Condition 1: (Cont'd.)

1.0	0.5	0.5	1.0
-26.40	+26.10	-2.0	+2.0
		-1.0	-2.0
	-11.55	-11.55	
+26.40	+13.20		
	-6.60	-6.60	
	+21.15	-21.15	

RELATIVE STIFFNESS K
DISTRIBUTION FACTOR = K/ZK
FEM
UNLOCK JOINT 3, BALANCE
CARRY OVER
BALANCE
UNLOCK JOINT 1, BALANCE
CARRY OVER
BALANCE
SUMMATION OF MOMENTS

FREE BODY DIAGRAM 1

$$R_1 = \left[7.32(11.25) + 2.04(8.08) + 2.26(1.25) + 0.060(20.0)^2/2 - 21.15 \right] / 20.0 = 4.63 \text{ K}$$

$$M_{\max} = 4.63(8.75) - 0.060(8.75)^2/2 = 38.22 \text{ K}$$

FREE BODY DIAGRAM 2

$$R_3 = \left[21.15 - 0.060(20.0)^2/2 \right] / 20.0 = 0.46 \text{ K}$$

$$R_2 = 0.060(40.0) + 7.32 + 2.04 + 2.26 - 4.63 + 0.46 = 9.85 \text{ K}$$

BY INSPECTION, MAX. SHEAR OCCURS WHEN MAX. WHEEL IS @ R_2

CONSERVATIVELY,

$$V_y = 0.060(20.0) + 7.32 + 2.04 \left(\frac{16.83}{20.0} \right) + 2.26 \left(\frac{10.0}{20.0} \right) = 11.37 \text{ K}$$

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HANFORD

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DESIGN ANALYSIS

Client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *J.C. Mackay*

Revised

By

Load Condition 2: Monorail loaded at maximum with load centered on new slave girder. Monorail is positioned at end of bridge girder. Also, 2 ton hoist is unloaded, and positioned at end of bridge girder.

PLAN

NOTE: MONORAIL & TROLLEY ARE
 POSITIONED @ STOPS

CALCULATE MAX. WHEEL LOADS FOR POINTS A, B, & C - SEE PAGE 2 FOR DESIGN LOADING
 & MAX. WEIGHTS

POINT AGIRDER +
END TRUCKMONORAIL ASSY
- HOISTHOIST &
TROLLEY

IMPACT

$$\text{MAX.} = \left[\left[0.070(31.0) + 0.478 \right] \frac{1}{2} + (1.163 - 0.411) \left(\frac{27.5}{31.0} \right) + 1.145 \left(\frac{1}{2} \right) \left(\frac{28.75}{31.0} \right) \right] 1.10 = 2.77^k$$

POINT B

$$\text{MAX.} = \left[\left[0.070(31.0) + 0.478 \right] \frac{1}{2} + 1.145 \left(\frac{1}{2} \right) \left(\frac{28.75}{31.0} \right) \right] 1.10 = 2.04^k$$

POINT C

$$\text{MAX.} = 0.070(31.0) \frac{1}{2} + 0.390 + \left[4.0(1.25) + 1.163 \right] \left(\frac{27.5}{31.0} \right) = 6.85^k \quad \leftarrow \text{USE}$$

OR

$$\text{MAX.} = \left[0.070(31.0) \frac{1}{2} + 0.390 + \left[4.0 + 1.163 \right] \left(\frac{27.5}{31.0} \right) \right] 1.10 = 6.56^k$$

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *C. Mackey*

Revised

By

Load Condition 2: (Cont'd.)**CALCULATE C.G. OF WHEEL LOADS:**

$$C.G. = \frac{2.0(3.17) + 6.85(10.0)}{2.77 + 2.04 + 6.85} = 6.43'$$

SLAVE GIRDER

By inspection, max. moment in crane rail will occur under the loaded W14x61 when the loads are positioned as shown below. Ref. AISC manual, p. 2-310, "General Rules for Simple Beams Carrying Moving Concentrated Loads"

Use moment distribution to calculate maximum bending moments in crane rail. Conservatively, treat crane rail as a two span continuous beam.

DISTANCE FROM #

$$\text{Column To Point A} = \{20.0 - [10.0 + 6.43]\} / 2 = 1.785'$$

CALCULATE FIXED END MOMENTS @ POINTS 1, 2 & 3 - Ref. AISC Manual, p. 2-301
 Load Cases 15 & 17.

$$M_{1L} = - \left[\frac{0.060(20.0)^2}{12} + \frac{2.77(1.785)(18.215)^2}{20.0^2} + \frac{2.04(4.955)(15.045)^2}{20.0^2} + \frac{6.56(11.785)(8.215)^2}{20.0^2} \right] = -24.96 \text{ } ^1\text{-K}$$

$$M_{2L} = \frac{0.060(20.0)^2}{12} + \frac{2.77(18.215)(1.785)^2}{20.0^2} + \frac{2.04(15.045)(4.955)^2}{20.0^2} + \frac{6.56(8.215)(11.785)^2}{20.0^2} = +23.00 \text{ } ^1\text{-K}$$

$$M_{2R} = - \frac{0.060(20.0)^2}{12} = -2.0 \text{ } ^1\text{-K}$$

$$M_{3L} = \frac{0.060(20.0)^2}{12} = +2.0 \text{ } ^1\text{-K}$$

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 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *C. Mackay*

Revised

By

Load Condition 2: (Cont'd.)

1.0	0.5	0.5	1.0
-24.86	+23.00	-2.0	+2.0
		-1.0	-2.0
	-10.0	-10.0	
+24.86	+12.43		
	-6.215	-6.215	
	-19.215	-19.215	

REL. SPAN STIFF. K DISTRIB. FACTOR = $K/2K$

FEM

BALANCE, CARRY OVER

BALANCE

BALANCE, CARRY OVER

BALANCE

SUMMATION OF MOMENTS

FREE BODY DIAGRAM 1

$$R_1 = \left[0.060(20.0)^2/2 + 2.77(18.215) + 2.04(15.045) + 6.56(8.215) - 19.215 \right] / 20.0 = 6.39 \text{ k}$$

$$M_{max} = 6.39(11.785) - 0.060(11.785)^2/2 - 2.77(10.0) - 2.04(6.83) = 29.51 \text{ k-ft}$$

FREE BODY DIAGRAM 2

$$R_3 = \left[19.215 - 0.060(20.0)^2/2 \right] / 20.0 = 0.36 \text{ k}$$

$$R_2 = 0.060(40.0) + 2.77 + 2.04 + 6.56 - 6.39 + 0.36 = 7.74 \text{ k}$$

By Inspection, Max. Stress Occurs When Max. Wheel Load Is @ R_2

$$\text{CONSERVATIVELY, } V_y = 0.060(20.0) + 6.56 + 2.04 \left(\frac{13.17}{20.0} \right) + 2.77 \left(\frac{10.0}{20.0} \right) = 10.49 \text{ k}$$

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Date 12/05/95

By Mark Axup

Checked 12/21/95

By *C. Mackey*

Revised

By

Load Condition 3: Bridge crane hoist loaded at maximum with load positioned at end of bridge girder. Monorail is positioned at end of bridge girder, unloaded.

PLU

NOTE: MONORAIL & TROLLEY ARE
 POSITIONED @ STOPS

CALCULATE MAX. WHEEL LOADS FOR POINTS A, B & C - SEE PAGE 2 FOR DESIGN LOADINGS

POINT A

BRIDGE GIRDER	SUB TRUCK	TROLLEY HOIST	TEST LOAD	MONORAIL ASSY.	MATL. WEIGHTS
$0.070 (31.0) + 0.478$	$1.145 + 4.0 (1.25)$	$\frac{28.75}{31.0}$	$1.163 \left(\frac{27.5}{31.0} \right)$		

$$\text{Max.} = \left[0.070 (31.0) + 0.478 \right] / 2 + \left[1.145 + 4.0 (1.25) \right] / 2 \left(\frac{28.75}{31.0} \right) + 1.163 \left(\frac{27.5}{31.0} \right) = 5.21^k$$

OR

$$\text{Max.} = \left[\left[0.070 (31.0) + 0.478 \right] / 2 + \left[1.145 + 4.0 \right] / 2 \left(\frac{28.75}{31.0} \right) + 1.163 \left(\frac{27.5}{31.0} \right) \right] 1.10 = 5.22^k \leftarrow \text{USE}$$

POINT B

$$\text{Max.} = \left[0.070 (31.0) + 0.478 \right] / 2 + \left[1.145 + 4.0 (1.25) \right] / 2 \left(\frac{28.75}{31.0} \right) = 4.17^k \leftarrow \text{USE}$$

OR

$$\text{Max.} = \left[\left[0.070 (31.0) + 0.478 \right] / 2 + \left[1.145 + 4.0 \right] / 2 \left(\frac{28.75}{31.0} \right) \right] 1.10 = 4.08^k$$

POINT C

$$\text{Max.} = \left[0.070 (31.0) / 2 + 0.300 + (1.163 - 0.411) \left(\frac{27.5}{31.0} \right) \right] 1.10 = 2.26^k$$

KAISER ENGINEERS
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DESIGN ANALYSIS

Client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *S. Mackey*

Revised

By

Load Condition 3: (Cont'd.)

CALCULATE C.G. OF POINT LOADS A, B, & C

$$C.G. = \frac{4.17(3.17) + 2.26(10.0)}{5.22 + 4.17 + 2.26} = 3.07'$$

By inspection, max. moment in crane rail will occur under the loaded W14x61, at point B, when the loads are positioned as shown below. Ref. AISC manual, p. 2-310, "General Rules for Simple Beams Carrying Moving Concentrated Loads"

Use moment distribution to calculate maximum bending moments in crane rail. Conservatively, treat crane rail as a two span continuous beam.

DISTANCE FROM E
 COLUMN TO POINT A : $[20.0 - (3.17 + 3.07)]/2 = 6.88'$

CALCULATE FIXED END MOMENTS @ POINTS 1, 2 & 3 - REF. AISC MANUAL, p. 2-301
 LOAD CASES 15 & 17

$$M_{1L} = - \left[\frac{0.060(20.0)^2}{12} + \frac{5.22(6.88)(13.12)}{20.0^2} + \frac{4.17(10.05)(9.95)}{20.0^2} + \frac{2.26(16.88)(3.12)}{20.0^2} \right] = -28.76 \text{ k-ft}$$

$$M_{2L} = \frac{0.060(20.0)^2}{12} + \frac{5.22(13.12)(6.88)}{20.0^2} + \frac{4.17(9.95)(10.05)}{20.0^2} + \frac{2.26(3.12)(16.88)}{20.0^2} = +25.60 \text{ k-ft}$$

$$M_{2R} = \frac{0.060(20.0)^2}{12} = -2.0 \text{ k-ft}$$

$$M_{3L} = \frac{0.060(20.0)^2}{12} = +2.0 \text{ k-ft}$$

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Client Westinghouse
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 and 2 Ton Bridge Crane

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By *S. Mackey*

Location 305 Bldg.

Revised

By

Load Condition 3: (Cont'd.)

1.0	0.5	0.5	1.0
-29.76	+25.60	-2.0	+2.0
		-1.0	-2.0
	-11.30	-11.30	
+29.76	+14.38		
	-7.19	-7.19	
	<u>+21.49</u>	<u>-21.49</u>	

REL. SPAN STIFF. K DISTRIB. FACTOR = K/EK

FEM

BALANCE, CARRY OVER

BALANCE

BALANCE, CARRY OVER

BALANCE

SUMMATION OF MOMENTS

FREE BODY DIAGRAM 1

$$R_1 = \left[0.060(20.0)^2/2 + 5.22(13.12) + 4.17(9.95) + 2.26(3.12) - 21.49 \right] / 20.0 = 5.38 \text{ k}$$

$$M = 5.38(6.88) - 0.060(6.88)^2/2 = 35.57 \text{ k-ft}$$

FREE BODY DIAGRAM 2

$$R_3 = [21.49 - 0.060(20.0)^2/2] / 20.0 = 0.47 \text{ k}$$

$$R_2 = 0.060(40.0) + 5.22 + 4.17 + 2.26 - 5.38 - 0.47 = 9.14 \text{ k}$$

By Inspection, max shear occurs when max wheel load is @ R_2

CONSERVATIVELY, $V_y = 0.060(20.0) + 5.22 + 4.17 \left(\frac{16.83}{20.0} \right) + 2.26 \left(\frac{10.0}{20.0} \right) = 11.06 \text{ k}$

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DESIGN ANALYSIS

client Westinghouse
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 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 1/09/96

Checked 1/9/96

Revised

By Mark Axup

By *Mark Axup*

By

Loading Conclusion: Load condition 1, which has all the load on the monorail assy., controls as the worst load case for analyzing the crane rail and also the column attachment. Load condition 3, which has all the load on the bridge crane, is less than load condition 1. Therefore, any combination of loading between the monorail and the bridge crane will be less critical than when the monorail is fully loaded.

Lateral loading: Per section A4.3 of the AISC manual, p. 5-29, the lateral force for analysis is 20% of the sum of the lifted load and crane trolley only. Load conditions 1 and 2 have the monorail loaded. The monorail is to be used for a K-basin mockup where the lifted load will only be moved parallel to the crane rails. Furthermore, the motors for the monorail, both the end trucks and the monorail hoist, are at very low speed ratings of 16 fpm and also are equipped with soft starts and stops. Therefore, the monorail load conditions will not impose a lateral load to the crane rails.

Use load condition 3 to calculate the lateral bending moment in the crane rails.

Calculate Maximum Lateral Wheel Loads for Points A & B

Since the crane rail is rigidly restrained at top and bottom flanges at it's connection to the building columns, consider beam as being fixed at both ends for weak axis bending. Conservatively, calculate maximum bending moment by applying entire load at center of span.

$$M_{\text{LATERAL}} = \frac{0.020}{2} [40(1.25) + 1.145] / 2 = 0.307^{\text{K}}$$

$$M_{\text{LATERAL}} = \frac{PL}{8} = \frac{2(0.307)(20.0)}{8} = 1.54^{\text{K}}$$

Compare Maximum Lateral Wheel Loads due to Impact to Lateral Loading due to Seismic

SINCE THE LIFTED LOAD IS SUPPORTED AS A PENDULUM, THERE IS NOT A RIGID MEMBER TO LATERALLY TRANSFER A SEISMIC FORCE DUE TO THIS MASS INTO THE CRANE RAILS.

$$M_{\text{LATERAL}} \text{ DEAD LOAD @ CRANE RAILS} = \frac{\text{TROLLEY} + \text{END TRUCK} + \text{WHEELS}}{2} = \frac{1.145 + 0.478 + 1.163}{2} = 1.393^{\text{K}}$$

$$\text{PER SEC 4.1, PEAK SEISMIC FOR S.C. 3} = 0.25(1.5) = 0.375$$

$$\text{LATERAL SEISMIC PER CRANE RAIL} = 0.375(9.30)/2 = 1.74^{\text{K}} > 0.307(2) = 0.614^{\text{K}} \text{ CALCULATED ABOVE}$$

MUST CHECK SEISMIC

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Client Westinghouse
 Subject Crane Rail Analysis for 2 Ton Monorail
 and 2 Ton Bridge Crane
 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00
 Date 12/05/95 By Mark Axup
 Checked 1/9/96 By *[Signature]*
 Revised By

Crane Rail Stress Analysis:DESIGN LOADING:

(FROM PREVIOUS PAGES 2 THROUGH 11)

$$\left. \begin{array}{l} M_x = 38.22 \text{ }^1\text{-K} \\ V_y = 11.37 \text{ }^1\text{-K} \\ M_x = 35.59 \text{ }^1\text{-K} \\ V_y = 1.54 \text{ }^1\text{-K} \end{array} \right\} \begin{array}{l} \text{SEE LOAD CONDITION 1} \\ \text{COMBINED LOADING} \\ \text{SEE LOAD CONDITION 3} \end{array}$$

W18x40 SECTION PROPERTIES

$$\left. \begin{array}{l} L = 11.80 \text{ }^1\text{ft} \\ d = 17.90 \text{ }^1\text{in} \\ t_w = 0.315 \text{ }^1\text{in} \\ b_f = 6.015 \text{ }^1\text{in} \\ t_f = 0.525 \text{ }^1\text{in} \\ F_t = 1.52 \text{ }^1\text{in} \\ d/A_f = 5.67 \end{array} \right\} \begin{array}{l} I_x = 612 \text{ }^1\text{in}^4 \\ S_x = 68.4 \text{ }^1\text{in}^3 \\ I_y = 19.1 \text{ }^1\text{in}^4 \\ b_c = 6.30 \text{ }^1\text{ft} \\ L_u = 8.20 \text{ }^1\text{ft} \\ S_y = \frac{I_y/2}{b_f/2} = \frac{19.1}{6.015} = 3.18 \text{ }^1\text{in}^3 \end{array} \right\} \begin{array}{l} \text{2-61, AISC} \\ \text{MAN.} \end{array}$$

TOP HALF OF BEAM ONLY

$$L/r_t = \frac{20.0(12)}{1.52} = 157.89 > \sqrt{\frac{510 \times 10^3 C_b}{F_y}} = 119.02$$

$$F_{bx} = \frac{170 \times 10^3 C_b}{(L/r_t)^2} = \frac{170 \times 10^3 (1.0)}{(157.89)^2} = 6.82 \text{ }^1\text{ksi} \quad \text{EQUATION F1-7, AISC MAN. p. 5-47}$$

OR

$$F_{bx} = \frac{12 \times 10^3 C_b}{L d/A_f} = \frac{12 \times 10^3 (1.0)}{20.0(12)(5.67)} = 8.82 \text{ }^1\text{ksi} \rightarrow \text{USE} \quad \text{EQ F1-8, AISC MAN. p. 5-47}$$

$$F_{by} = 0.75 F_y = 0.75 (36.0) = 27.0 \text{ }^1\text{ksi} \quad \text{EQ F2-1, AISC MAN. p. 5-48}$$

$$f_{bx} = \frac{38.22(12)}{68.4} = 6.71 \text{ }^1\text{ksi} < 8.82 \text{ }^1\text{ksi} \quad \text{OK}$$

$$f_{vy} = \frac{11.37}{(17.90)(0.315)} = 2.02 \text{ }^1\text{ksi} < F_v = 0.4 F_y = 14.4 \text{ }^1\text{ksi} \quad \text{OK}$$

CHECK COMBINED LOADING

$$f_{bx} = \frac{35.59(12)}{68.4} = 6.24 \text{ }^1\text{ksi} \quad f_{by} = \frac{1.54(12)}{3.18} = 5.81 \text{ }^1\text{ksi}$$

$$I.R. = \frac{6.24}{8.82} + \frac{5.81}{27.0} = 0.92 < 1.0 \quad \text{OK}$$

CHECK COMBINED LOADING BY RATIO

$$I.R. = \frac{6.24}{8.82} + \frac{5.81 \left(\frac{1.74}{0.614} \right)}{27.0} = 1.32 < 1.33 \quad \text{OK}$$

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DESIGN ANALYSIS

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 Location 305 Bldg.

WO/Job No. LE112/KPCN P97J00

Date 12/05/95

By Mark Axup

Checked 12/21/95

By J.C. Mackey

Revised

By

Check Attachment of Crane Rail to Column:CONFIGURATION: DESIGN LOAD:

$$V_y = 11.37^k \text{ SEE LOAD CONDITION 1 max}$$

CONSERVATIVELY, CALCULATE MOMENT @
 E OF COLUMN.

$$M = 11.37(1.32) = 15.0^{k-ft} \text{ max}$$

CHECK WELD: TREAT WELD AS A LINE
 REF. BRIDGEMAN, DESIGN OF
 WELDED STRUCTURES.

WELD PROPERTIES:

$$A = 2 \times 18 = 36 \text{ in}$$

CONSERV.

$$S = 2bd + \frac{d^2}{3}$$

$$= 2(6.0)(18.0) + \frac{18.0^2}{3} = 324.0 \text{ in}^2$$

WELD STRESS:

$$f_w = \left[\left(\frac{11.37}{36.0} \right)^2 + \left(\frac{15.0(12)}{324.0} \right)^2 \right]^{1/2} = 0.639 \text{ ksi}$$

$$w_{REQD} = \frac{0.639}{0.3(70.0)(0.707)} = 0.04" < 3/16" \text{ OK}$$

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Date 12/05/95 By Mark Axup

Checked 12/21/95

By *M. Axup*

Revised

By

Check Attachment of Crane Rail to Column: (Cont'd.)CHECK REQUIREMENTS OF LOCAL FLANGE BENDING:

REF. SECTION K1.2, p. 5-50, OF AISC MANUAL

$$P_{bf} = \left[\frac{15.0 (12)}{17.90 - 0.525} \right] \frac{5}{3} = 17.27 \text{ K}$$

$$t_f = 0.4 \sqrt{\frac{17.27}{36.0}} = 0.277" < 0.645" \text{ OK}$$

COL. REQ'D

CHECK LOCAL WEB BUCKLING:

REF. SECT. K1.3, p. 5-21 AISC MAN.

h = 1.375", N = t_f = 0.525", t_w = 0.430"

$$R = \frac{17.27}{t_w (N + 5t_f)} = \frac{17.27}{0.430 [0.525 + 5(1.375)]}$$

$$= 5.43 \text{ ksi} < 23.76 \text{ ksi} \text{ OK}$$

CRANE RAIL GIRDED, W18x40, TO COL. W16x58, CONNECTION IS OKCHECK COLUMN FOR COMBINED LOADING OF 2 TON BRIDGE CRANE & MONORAILAND 5 TON BRIDGE CRANE

DUE TO BRIDGE SIMILARITY FOR 2 TON & 5 TON BRIDGE CRANE, CONSERVATIVELY
 CALCULATE MAX. REACTION FOR 5 TON BRIDGE CRANE BASED ON MAX. REACTION
 FOR 2 TON BRIDGE CRANE + 3 TONS MULTIPLIED BY 1.25

$$V_y = (11.37 + 6.0) 1.25 = 21.7 \text{ K}$$

10 TON MAX

$$V_x = \frac{0.20}{2} (10.0 + 2.0) = 1.2 \text{ K}$$

10 TON

ESTIMATED WEIGHT OF
5 TON BRIDGE TROLLEY

=

None: K = 1.0

Column Loading

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

Client Westinghouse
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Date 12/05/95

Checked 1/9/96

Revised

By Mark Axup

By *S. Mackey*

By

Check Column For Combined Loading: (Cont'd.)

W16x54 PROPERTIES

$$A = 17.04 \text{ in}^2$$

$$d/A_f = 2.91$$

$$S_x = 94.1 \text{ in}^3$$

$$r_x = 6.62 \text{ in}$$

$$r_y = 1.98 \text{ in}$$

$$L = 24.0 \text{ ft}$$

unbraced

$$f_{b_n} = \frac{22.2 (12)}{94.1} = 283 \text{ ksi}$$

$$F_{b_n} = \frac{12 \times 10^3 C_b}{\lambda d/A_f} = \frac{12 \times 10^3 (1.0)}{24.0 (12) (2.91)} = 14.32 \text{ ksi}$$

$$\frac{K L_y}{r_y} = \frac{1.0 (24.0) 12}{1.98} = 153.19 \longrightarrow \bar{F}_a = 6.36 \text{ ksi (BY INTERPOLATION, SEE TABLE C-36, p. 3-16, AISC MANUAL)}$$

$$\frac{K L_x}{r_x} = \frac{1.0 (24.0) 12}{6.62} = 43.50 \quad \bar{F}_x = \frac{12 \pi^2 29 \times 10^3}{23 (43.50)^2} = 78.92 \text{ ksi}$$

$$f_a = \frac{33.07}{17.04} = 1.94 \text{ ksi} \quad \frac{f_a}{\bar{F}_a} = \frac{1.94}{6.36} = 0.31 > 0.15$$

 $\therefore$ Check Beam Interpolation Using Eq. H1-1

p. 5-54, AISC Manual

$$I.R. = \frac{f_a}{\bar{F}_a} + \frac{C_{mn} f_{b_n}}{\left[1 - \frac{f_a}{\bar{F}_x}\right] \bar{F}_{b_n}} = \frac{1.94}{6.36} + \frac{1.0 (283)}{\left[1 - \frac{1.94}{78.92}\right] 14.32} = 0.51 < 1.0 \quad \text{OK}$$

Conclusion:

This column analysis of the building crane structure ~~verifies the validity~~ ^{EXTENDS THE CONCLUSION} of the letter from Gary Wagenblast, dated 8/23/83 (see attachment 2). In the letter, Mr. Wagenblast states that the structure was analyzed for a 10 ton bridge crane for the top rails and a manipulator bridge crane (of unknown rated capacity) for the lower rails. The letter only identifies crane rail and beam to column deficiencies that would need to be addressed for a much higher load combination than is required for this modification and is addressed in this calculation. Since the crane rail and its connections are qualified in this calculation, no further building analysis is required.

mtl
1-9-96

Attachment 1:

Vendor Bridge Crane And Monorail Assembly Calculations
Including Kaiser Alternate Check Calculations

ATTACHMENT 1 CONTENTS

page	item
1-11	Bartholomew Co. HCVF Crane Recalculation
12	2T monorail column (est), Bartholomew Co.
13-19	steel member data
20-28	ICF KH alternate check calculations

THE BARTHOLOMEW COMPANY, INC.

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REVISION — DATE 5-3-91

JOB NUMBER 9366

PREPARED BY H.G.O.T.

DOCUMENT No. 950905

CUSTOMER WEST. - HANFORD

TITLE H.C.V.E. CRANE RE-ALC.

1.0 ORIGINAL HOIST/TROLLEY/TUBES + 31.5
 (FROM C.S. CALC.)

LESS: HOIST MOTOR + 470
 CABLE REEL + 100
 TELE TUBES + 154
 MANIPULATOR ADAPTOR + 100

PLUS: NEW MOTOR - 140
 NEW HOIST - 40

TOTAL HOIST/TROLLEY WT. NEW 114

2.0 GIRDER

LOAD = 4000 #
 15% IMPACT = 600 # (NISC.)
 HOIST/TROLLEY = 1145

(WHICH) 45745

$$1437 \# + \frac{12.38' \times 1437 \#}{15.5'} = 2585 \#$$

WHICH GIVE FOR 65860 # F.C.L. @ 31'
 (SEE ATTACHED TABLE)
 (INTERPOLATE)

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DOCUMENT No. 950905

3.0 END TRUCK (DUE TO EXIST HOIST/TROLLEY)

$$\frac{29.00' \times 5745"}{31.00'} = 5375\#$$

$$\begin{aligned} (2) \times 61\# \times 32' \times 1.5 &= 1952\# \text{ OF (2) SPOKES} \\ (2) \times 5.1\# \times 32' \times 1.5 &= 164\# \text{ OF (2) RAILS} \\ 6.01\# \times 32' \times 1.5 &= 97\# \text{ OF JACKSHAFI} \\ &= 84\# \text{ OF BRIDGE DRIVE} \\ 5\# \times 32' \times 1.5 &= 80\# \text{ OF FESTOON} \\ (1) \text{ END TRUCK} &= 4.78 \text{ INCL WHEELS} \\ \text{WHEELS 2} &= 8230\# \text{ TOTAL} \end{aligned}$$

$$4.10 \text{ WHEEL BEARING 2} = 4.15\#/\text{WHEEL}$$

$$\begin{aligned} \frac{29.00' \times 600}{31.00'} \times 1.5 &= 2058\#/\text{BEARING w IMPACT} \\ &= 281\# \text{ IMPACT} \\ &= 1777\#/\text{BEARING w IMPACT} \end{aligned}$$

WHEEL BEARING IS 210 W.D.G. "FAFNIR"

RADIAL LOAD RATING IS 7940# @ 33 1/3 R.P.
 FOR A 1500 HR L10 LIFE.

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5.0 5" ϕ WHEEL

5" ϕ WHEEL FOR A STANDBY SVL

B. LIGHT

C. MODERATE

15 5000 FOR 10,000 $\pm$

(SEE ATTACHED TABLE)

(INTERPOLATE)

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CUSTOMER

TITLE

6.0 MONORAIL

$$\begin{aligned} \text{LOAD} &= 4000\# \\ 15\% \text{ IMPACT} &= 600\# \quad (\text{AISC}) \\ \text{HOIST/TROLLEY} &= 411\# \\ &= 5011\# \end{aligned}$$

SB X 18.4 5000 FOR 7092# ECL @ 10'
 (SEE ATTACHED TABLE)

7.0 MONORAIL END TRUCK

$$\begin{aligned} \text{LOAD/IMPACT/HOIST/TROLLEY} &= 5011\# \\ 18.4\# \times 12' \times 1.5 &= 111 \quad 1/2 \text{ MONORAIL} \\ 5\# \times 12' \times 1.5 &= 30 \quad 1/2 \text{ FESTOON} \\ &= 288 \quad (1) \text{ COLUMN + BRK} \\ &= 323 \quad (1) \text{ END TRUCK} \\ 2 \overline{5.763\#} & \text{ TOTAL} \\ &= 2882\# / \text{WHEEL} \end{aligned}$$

8.0 GIRDER

$$\begin{aligned} 2882\# \times 10.5' \times 2882\# \\ 15.5' &= 433\# \\ & \text{E.C.L.} \end{aligned}$$

W14 X 61 6000 FOR 6586# ECL @ 31'
 (SEE ATTACHED TABLE)

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CUSTOMER

TITLE

9.0 EXIST END TRUCK (DUE TO NEW MONORAIL)
(ASSUMED EXIST HOIST/TROLLEY CENTRALIZED
& UNLOADED)

EXIST HOIST/TROLLEY

(2) X 61' X 32' X 1.5 = 1952 # 1/2 (2) GRDS SH.
2 X 5.1' X 32' X 1.5 = 164 # 1/2 (2) RAILS
1.01' X 32' X 1.5 = 97 # 1/2 U-SHAPED
5' X 32' X 1.5 = 80 # 1/2 FEETON
(1) END TRUCK = 478 INCL WHLS

WHEELS 2 / 3428 # TOTAL

1714 # / WHEEL

$\frac{27.50' \times 5767' \#}{31.00'} = 5113 \times \frac{4.67'}{6.17'} = 3670 \#$

LOAD DUE TO NEW MONORAIL

2 / 5584 # / WHEEL TOTAL

2792 # / E.S.G. W IMPACT

$\frac{27.50' \times 5220' \#}{31'} = 522 \times \frac{1.67'}{6.17'} = 403 \#$

IMPACT LOAD

2389 # / BRG. W/O IMPACT

FROM 4.0: BRG IS 210 WDDG "FAFENIR"
RADIAL LOAD RATING IS 29 #

@ 33 1/3 RPM, F22 1500 RPM
L10 LIFE

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PREPARED BY

TITLE

DOCUMENT No. 950905

10.0 SIMULTANEOUS GIRDER LOADS

FROM 2.0 = 2585# ECL W IMPACT

$150 + \frac{1233' \times 150}{15.5} = -270$ LESS IMPACT

2315# ECL W/O IMPACT

FROM 8.0 = 4935# ECL W IMPACT

$150 + \frac{1233' \times 150}{15.5} = -270$ LESS IMPACT

4565# ECL W/O IMPACT

$2315 + 4565 = \underline{6880}$

FROM 2.0 VS 6580# = SLIGHT O'LO.

∴ USE (1) PARTIAL LOAD.
 OR
 POSITION LOADS APART.

THE BARTHOLOMEW COMPANY, INC.

91 NEW YORK AVENUE WESTBURY, N.Y. 11590

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Fax (516) 333-0301

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REVISION B DATE 2/3/96

JOB NUMBER

PREPARED BY

DOCUMENT No. 950225-

CUSTOMER

TITLE

11.0 SIMULTANEOUS WHEEL BRG LOADS

FROM 4.0 1777 [#]

FROM 9.0 2389 [#]

4166 [#]

FROM 4.0 LESS THAN 7945 [#] ∴ OK

12.0 NEW 1/2 END TRUCK

FROM 9.0

5113 [#]

LOAD DUE TO
NEW MONORAIL

"

976 [#]

1/2 CYCLOPS

"

160 [#]

1/2 (2) FLEETON

NEW 1/2 END TRUCK

200 [#]

INCL WHEEL

6449 [#] TOTAL

∴ USE TOP RUN WHEEL ASSY OR
AT LEAST THIS RATING

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CUSTOMER

TITLE

13.0 END TRUCK STRUCT (SIMULTANEOUS LOADS)

FROM 3.0 4115# DUE TO ORIG. 2TON H/I
 FROM 9.0 5113# DUE TO NEW MONORAIL
 9228#

$$R_2 = \frac{P_1 a + P_2 (2-b)}{L} = \frac{4115 \times 13 + 9228 (74-13)}{74}$$

$$= 7984 \#$$

$$M_2 = R_2 b = 7984 \times 13 = 143,717 \#$$

$$S_{REQ} = \frac{M_2}{F} = \frac{143,717}{21,600} = 6.65$$

$$S_{XX} (2) MC 6 \times 12 = 12.48$$

OK

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REVISION C DATE 10 OCT

JOB NUMBER

CUSTOMER

PREPARED BY

TITLE

DOCUMENT No. 950905

14.0 MONORAIL SUPPORT COLS. (9366-P-205) ^{DRG.}

(FROM STOCK) 4" PIPE = 4 1/2 O.D. X .237 WALL
 X 10.79 #/ft X 3.17 □"

TENSION 3.17 □ X 58 KSI ÷ 5 SF = 36.7 K
 FROM 7.0 ACT LOAD IS 5763# - 323# (END TOLL)
 = 5440# i. OK

WELD 4.500 O.D. X .71 X .187" X 1.207 X 12 K
 = 22.43 K

FASTENERS (TENSION)
 (4) 3/4" φ A325
 (4) X 19.4 K = 77.5 K#
 (ALSO NINTH 7-3)

15.0 ORIG. GIRD CONN.

AREA
 (1) FLANGE 10" X 1.645"
 = 6.45 □"
 (2) FLANGE 10" X 1.500"
 = 5.00 □"
 (3) WEB 6.23" X .375"
 = 2.34 □"

13.79 □" TOTAL

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CUSTOMER _____

TITLE _____

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15.0 CONT.

$$58 \text{ KSI} \times .75 = 43.5 \text{ KSI SHEAR} \times 13.79 \square''$$

$$= 599.9 \text{ K} \div 5 \text{ SF} = \underline{119.9 \text{ K}}$$

SIMULTANEOUS LOADS

$$\text{FROM 3.0 } 8230\# - 478\# = 7752\# \div 2 = 3876\#$$

$$\text{" 9.0 } 6279\# - 478\# = 5801\# \div 2 = 2901\#$$

6777\# TOTAL

16.0 NEW GIRD CONN. (93CC-D-201) $\phi 4 \times 3/8$ $\phi 10 \times 1/2$

AREA:

$$(2) \text{ FLG. } 10" \times .500" = 10 \square''$$

$$(1) \text{ FLG. } 10" \times .435" = 4.35 \square''$$

$$(1) \text{ WEB } 4" \times .375 = 1.5 \square''$$

$$(1) \text{ WEB } 2.5" \times .375 = .94 \square''$$

18.89 \square'' TOTAL

BETTER THAN 15.0 ABOVE.

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PREPARED BY

TITLE

DOCUMENT No. 950905-

17.0 HOIST BLOCK

15 STD. "SATURN ENCL." 2 TON
FOR (4) PARTS OF $\frac{1}{4}$ " ϕ
CABLE. DWG ATTACHED
"SATURN" DWG CSA-107

18.0 HOISTING CABLE

15 $\frac{1}{4}$ " ϕ IWRC XIPS "PAULSEN
"GOLD CIRCLE SUPREME"

3.40 TONS BRK STRENGTH = 6300#
 $\times (4) \text{ CABLES} = 27,200\# \div 400$
 $= \underline{\underline{67.5 \text{ TONS}}}$

19.0 ITEMS NOT ADDRESSED HEREIN
WHICH ARE OK BY INSPECTION.

19.1 COLUMN BRAKING COMPONENTS:

19.1.1 COLUMN BRAKE ATTACH. π 'S & LIFT LL
DWG 9366-D-205

19.1.2 MONORAIL BRAKE ATTACH π 'S
DWG 9366-D-202

19.1.3 COLUMN BRACKS
DWG 9366-D-303-1,2,3
(ALL BOLTS ARE $3\frac{1}{4}$ " ϕ A325)

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REVISION A DATE 2058-95

JOB NUMBER 9366

PREPARED BY H.G.G.

DOCUMENT No. 95091B-

CUSTOMER WEST HANFORD

TITLE 2 TON MONO. COL (EST.)

(2) REQD

(4) 3/4" Ø BOLTS

5' x 12'

3/8" TYP

FOR INFO
ONLY DE MAN
ACTUAL

BRACE 11' x 4'

2) 4' x 2' x 1/4"

COL 13' x 15" = 195"
2) 5' 48" x 4" x 15" x 1/4" = 29"
3) BRACE 10' x 6' 1/4" = 64"

TOTAL 288"

WRIGHT®

STRUCTURAL BEAM DESIGN GUIDE AND SELECTION CHART FOR OVERHEAD CRANE RUNWAY SYSTEMS

Hoist & Crane Division

CRANE RAILS

U.S. STEEL &
 BETHLEHEM 135 LB.

BETHLEHEM 171 LB.

U.S. STEEL &
 BETHLEHEM 175 LB.

GUIDE FOR MAXIMUM BRIDGE AND TROLLEY WHEEL LOADINGS, LBS. (P)

C.M.A.A. Service Class	Wheel Dia. (D) Inches	RAIL SECTION									
		ASCE 20=	ASCE 25=	ASCE 30=	ASCE 40=	ASCE 60 & 70=	ASCE 80 & 85= Beth. 104= USS 105=	ASCE 100=	BETH. & USS 135=	BETH. & USS 175=	Bethlehem 171=
Class A1 & A2 INFREQUENT SERVICE Class B LIGHT SERVICE Class C MODERATE SERVICE P = 1600 WD	8	10800	12800	13610	16000						
	9	12150	14400	15310	18000	25200					
	10	13500	16000	17810	20000	28000					
	12		19200	20410	24000	32600	36000	40800			
	15			25510	30000	42000	45000	51000			
	18			30610	36000	50400	54000	61200			
	21				42000	58800	63000	71400	75600	105000	117600
	24					67200	72000	81600	86400	120000	134400
Class D HEAVY DUTY SERVICE P = 1400 WD	8	9450	11200	11900	14000						
	9	10630	12800	13390	15750	22050					
	10	11820	14000	14890	17500	24500					
	12		16800	17860	21000	29400	31500	35700			
	15			22320	26250	36750	39360	44630			
	18			26730	31500	44100	47250	53550			
	21				36750	51450	55130	62400	66150	91800	102900
	24					58800	63000	71400	75600	105000	117600
Class E SEVERE DUTY-CYCLE SERVICE P = 1200 WD	8	8100	9600	10200	12000						
	9	9120	10800	11400	13500	18900					
	10	10130	12000	12760	15000	21000					
	12		14400	15310	18000	25200	27000	30600			
	15			19120	22500	31500	33750	38250			
	18			22960	27000	37800	40500	45900			
	21				31500	44100	47250	53550	56700	78750	88200
	24					50400	54000	61200	64800	90000	100800
Effective Width of Rail Head (W) Inches (Top of head minus corner radius)		3.44	1.000	1.063	1.250	1.750	1.875	2.125	2.250	2.125	3.500

NOTE: The loading limits for Class E are also recommended wherever travel speeds exceed 400 f.p.m.

NOTES: Figures based on acceptable cast chilled iron, or rolled, forged, or cast steel properly heat treated. Figures do not include any allowance for impact. For unusual conditions, consideration must be given to other factors that are not included in the simple formula on which table figures are based.

FAFNIR

A-36

FAFNIR SINGLE ROW RADIAL BEARINGS

Light

200 Series

Load Ratings

Bearing Number	Radial Load Rating @ 33 1/3 RPM	Radial Load Ratings in Pounds at Various RPM Based on 1500 Hours L ₁₀ Life											
		30	100	200	300	500	700	1200	1500	1800	2400	3400	5000
200K Non-Filling Slot Type													
200K	1030	900	715	565	495	418	343	312	290	272	248	216	194
201K	1180	1030	815	645	565	477	392	356	330	311	283	247	221
202K	1340	1170	930	735	645	545	446	405	376	354	322	281	252
203K	1820	1590	1260	1000	875	740	605	550	510	482	438	382	343
204K	2210	1930	1530	1220	1060	895	740	670	620	585	530	465	416
205K	2420	2120	1680	1330	1160	985	810	735	680	640	580	510	456
206K	2570	3120	2480	1970	1720	1450	1190	1080	1000	945	860	750	670
207K	4620	4040	3200	2540	2220	1870	1540	1400	1300	1220	1110	970	870
208K	5640	4920	3910	3100	2710	2280	1880	1710	1580	1490	1350	1180	1060
209K	5650	4940	3920	3110	2720	2290	1880	1710	1590	1500	1360	1190	1060
210K	6060	5290	4200	3330	2910	2460	2020	1840	1700	1600	1460	1270	1140
211K	7490	6540	5190	4120	3600	3040	2500	2270	2100	1980	1800	1570	1400
212K	9050	7910	6280	4980	4350	3670	3020	2740	2540	2400	2180	1900	1700
213K	9890	8640	6860	5440	4760	4010	3300	3000	2780	2620	2380	2080	1860
214K	10730	9380	7440	5910	5160	4350	3580	3250	3020	2840	2580	2250	2000
215K	10710	9360	7430	5890	5150	4340	3570	3240	3010	2830	2580	2250	2000
218K	16600	14500	11500	9110	7960	6710	5520	5010	4650	4380	3980	3450	3050
200PP	805	705	560	444	388	327	269	244	227	213	194	169	152
201NPP	1050	920	730	580	505	427	351	319	296	279	253	221	198
202NPP	1180	1030	815	645	565	476	392	356	330	311	283	247	221
203PP	1650	1440	1140	910	790	665	550	500	465	435	395	345	310
204PP	2210	1930	1530	1220	1060	895	740	670	620	585	530	465	416
205PP	2420	2120	1680	1330	1160	985	810	735	680	640	580	510	456
206PP	2570	2940	2330	1850	1620	1360	1120	1020	945	890	810	705	635
209NP	5670	4950	3930	3120	2720	2300	1890	1720	1590	1500	1360	1190	1070
210PP	6060	5290	4200	3330	2910	2460	2020	1840	1700	1600	1460	1270	1140
211NPP	7520	6570	5210	4140	3610	3050	2510	2280	2110	1990	1810	1580	1400
200W Filling Slot Type													
202W	1700	1480	1180	935	815	690	565	515	477	449	408	357	319
204W	3000	2620	2080	1650	1440	1210	1000	905	840	790	720	630	565
205W	3460	3020	2400	1900	1660	1400	1150	1050	970	915	830	725	650
206W	4680	4090	3250	2580	2250	1900	1560	1420	1320	1240	1120	985	880
207W	6060	5290	4200	3330	2910	2460	2020	1830	1700	1600	1460	1270	1140
208W	7210	6300	5000	3970	3460	2920	2400	2180	2030	1910	1730	1510	1360
209W	7590	6630	5260	4180	3650	3080	2530	2300	2130	2010	1820	1590	1430
210W	7940	6930	5500	4370	3820	3220	2650	2400	2230	2100	1910	1670	1490
211W	9370	8190	6500	5160	4510	3800	3120	2840	2640	2480	2250	1970	1750
212W	11900	10400	8220	6530	5700	4810	3950	3590	3340	3140	2850	2490	2190
213W	14100	12300	9770	7760	6780	5720	4700	4270	3960	3730	3390	2960	2590
214W	14700	12800	10200	8080	7060	5950	4900	4450	4130	3880	3530	3080	2690
215W	15300	13300	10600	8400	7340	6190	5090	4620	4290	4040	3670	3200	2790
216W	17900	15400	12400	9830	8590	7240	5960	5410	5020	4730	4290	3760	3290
217W	19800	17300	13700	10900	9530	8030	6600	6000	5570	5240	4760	4180	3690
218W	22600	19800	15700	12500	10900	9180	7550	6860	6370	5990	5440	4780	4200
219W	25700	22400	17800	14100	12300	10400	8540	7770	7220	6790	6170	5400	4750
220W	28900	25200	20000	15900	13900	11700	9620	8740	8110	7640	6940	6050	5300
221W	30060	26200	20800	16500	14450	12200	10000	9100	8450	7950	7250	6290	5500
222W	34000	29700	23600	18700	16400	13800	11300	10300	9570	9000	8200	7140	6290
224W-3	33600	29300	23300	18500	16100	13600	11200	10200	9440	8880	8080	7020	6170
226W-3	39400	34400	27300	21700	18900	16000	13100	11900	11100	10400	9500	8340	7390

Section Designation Number	Depth of Section	Weight per Foot	Area of Section	Web Thickness	Flange Width	Flange Average Thickness	Area of Flange	$\frac{d}{A_f}$	Elastic Properties Axis X-X			Flange Nominal Thickness at Toe	Radius at Toe	Radius at Fillet	Usual Flange Gage	Maximum Flange Fastener
	d		A	t_w	b_f	t_f	A_f		I	S	r	t_f	r_t	r_f	g	f
	in.	lb.	in. ²	in.	in.	in.	in. ²		in. ⁴	in. ³	in.	in.	in.	in.	in.	in.
S5x12.5	6.0	12.5	3.67	0.232	3.532	0.259	1.196	5.02	22.1	7.37	2.45	$\frac{1}{4}$	.14	.33	2	$\frac{3}{4}$
S6x17.25	6.0	17.25	5.07	0.465	3.565	0.359	1.280	4.69	26.3	8.77	2.28	$\frac{1}{4}$	.14	.33	2	$\frac{3}{4}$
S7x15.3	7.0	15.3	4.50	0.252	3.662	0.392	1.435	4.88	36.7	10.5	2.86	$\frac{1}{4}$	.15	.35	2 $\frac{1}{4}$	$\frac{3}{4}$
S6x18.4	8.0	18.4	5.41	0.271	4.001	0.425	1.700	4.70	57.6	14.4	3.26	$\frac{1}{4}$	.16	.37	2 $\frac{1}{4}$	$\frac{3}{4}$
S6x23	8.0	23.0	6.77	0.441	4.171	0.425	1.773	4.51	64.9	16.2	3.10	$\frac{1}{4}$	.16	.37	2 $\frac{1}{4}$	$\frac{3}{4}$
S10x25.4	10.0	25.4	7.46	0.511	4.661	0.491	2.286	4.37	124	24.7	4.07	$\frac{3}{16}$	.19	.41	2 $\frac{3}{4}$	$\frac{3}{4}$
S10x35	10.0	35.0	10.3	0.594	4.944	0.491	2.428	4.12	147	29.4	3.78	$\frac{3}{16}$	.19	.41	2 $\frac{3}{4}$	$\frac{3}{4}$
S12x31.8	12.0	31.8	9.35	0.350	5.000	0.544	2.720	4.41	218	36.4	4.83	$\frac{3}{8}$	.21	.45	3	$\frac{3}{4}$
S12x35	12.0	35.0	10.3	0.428	5.078	0.544	2.762	4.34	229	38.2	4.72	$\frac{3}{8}$	.21	.45	3	$\frac{3}{4}$
S12x40.8	12.0	40.8	12.0	0.462	5.252	0.659	3.461	3.46	272	45.4	4.77	$\frac{7}{16}$	.28	.56	3	$\frac{3}{4}$
S12x50	12.0	50.0	14.7	0.687	5.477	0.659	3.609	3.32	305	50.8	4.55	$\frac{7}{16}$	.28	.56	3	$\frac{3}{4}$
S15x42.9	15.0	42.9	12.6	0.411	5.501	0.622	3.472	4.38	447	59.6	5.95	$\frac{7}{16}$	.25	.51	3 $\frac{1}{2}$	$\frac{3}{4}$
S15x50	15.0	50.0	14.7	0.550	5.640	0.622	3.508	4.28	486	64.8	5.75	$\frac{7}{16}$	.25	.51	3 $\frac{1}{2}$	$\frac{3}{4}$
S18x54.7	18.0	54.7	16.1	0.461	6.001	0.691	4.147	4.34	804	89.4	7.07	$\frac{7}{16}$	.28	.56	3 $\frac{1}{2}$	$\frac{3}{4}$
S18x70	18.0	70.0	20.6	0.711	6.251	0.691	4.319	4.17	926	103	6.71	$\frac{7}{16}$	.28	.56	3 $\frac{1}{2}$	$\frac{3}{4}$
S20x65.4	20.0	65.4	19.2	0.500	6.250	0.789	4.931	4.05	1180	118	7.84	$\frac{7}{16}$	.30	.60	3 $\frac{1}{2}$	$\frac{3}{4}$
S20x75	20.0	75.0	22.1	0.641	6.391	0.789	5.042	3.96	1280	128	7.60	$\frac{7}{16}$	.30	.60	3 $\frac{1}{2}$	$\frac{3}{4}$
S20x85	20.0	85.0	25.0	0.653	7.053	0.916	6.460	3.09	1520	152	7.79	$\frac{3}{4}$	.36	.70	4	1
S20x95	20.0	95.0	27.9	0.800	7.200	0.916	6.595	3.03	1610	161	7.60	$\frac{3}{4}$	.36	.70	4	1
S24x79.9	24.0	79.9	23.5	0.501	7.001	0.871	6.096	3.94	2110	175	9.47	$\frac{3}{4}$	.30	.60	4	1
S24x90	24.0	90.0	26.5	0.624	7.124	0.871	6.205	3.67	2250	187	9.22	$\frac{3}{4}$	.30	.60	4	1
S24x100	24.0	100.0	29.4	0.747	7.247	0.871	6.312	2.80	2390	199	9.01	$\frac{3}{4}$	.30	.60	4	1
S24x105.9	24.0	105.9	31.1	0.625	7.875	1.102	8.678	2.77	2830	236	9.53	$\frac{13}{16}$	.30	.60	4	1
S24x120	24.0	120.0	35.3	0.798	8.048	1.102	8.869	2.71	3030	252	9.26	$\frac{13}{16}$	.30	.60	4	1

NOTE:

Prior to approximately 1940, thirty (30) sizes of American standard shapes were rolled in addition to those tabulated above. These beams are now considered obsolete, but may still be found in older installations. For such cases, where the obsolete shape designation is known or actual measurements of the beam indicate it is not one of the above standard shapes, refer to "Steel Construction," 3rd Edition, or contact the York office.

REFERENCES:

- "Manual of Steel Construction," Seventh Edition, American Institute of Steel Construction, Inc., New York, 1970.
- "Steel Construction," Third Edition, American Institute of Steel Construction, Inc., New York, 1937.
- "Bethlehem Structural Shapes," 1975 Edition, Bethlehem Steel Corporation, Bethlehem, Pa., 1975.
- "Bethlehem Structural Shapes," 1962 Edition, Bethlehem Steel Corporation, Bethlehem, Pa., 1962.

AMERICAN STANDARD SHAPES (S SHAPES)

Lower flange loadings tabulated below are based on static load and do not consider service. Spacing between wheel centers must be equal to or greater than total flange width. All loading figures shown are in pounds.

S Section	Lower Flange Loading	Span in feet																				
		10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
6x12.5	2,540	2687 D	1826 D	1301 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
6x17.25	2,540	3183 D	2156 D	1528 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
7x15.3	3,040	4496 D	3074 D	2209 D	1641 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
8x18.4	3,600	7092 D	4867 D	3516 D	2631 D	2017 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
8x23	3,600	7977 D	5467 D	3942 D	2942 D	2248 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
10x25.4	4,830	14362 T	10584 D	7694 D	5807 D	4503 D	3561 D	2856 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-
10x35	4,830	17071 T	12510 D	9078 D	6835 D	5283 D	4161 D	3319 D	-	-	-	-	-	-	-	-	-	-	-	-	-	-
12x31.8	5,810	21193 T	16286 D	11951 D	9150 D	7229 D	5856 D	4640 D	4057 D	3465 D	-	-	-	-	-	-	-	-	-	-	-	-
12x35	5,810	22233 T	17153 D	12502 D	9648 D	7623 D	6175 D	5103 D	4288 D	3654 D	-	-	-	-	-	-	-	-	-	-	-	-
12x40.8	7,870	26478 T	21948 D	17007 D	12886 D	10045 D	7998 D	6471 D	5296 D	4371 D	-	-	-	-	-	-	-	-	-	-	-	-
12x50	7,870	29549 T	24533 D	19033 D	14407 D	11216 D	8915 D	7197 D	5875 D	4833 D	-	-	-	-	-	-	-	-	-	-	-	-
15x42.9	7,420	34747 T	28525 D	21886 D	16677 D	13122 D	10005 D	7443 D	6256 D	5331 D	4596 D	4004 D	3519 D	-	-	-	-	-	-	-	-	-
15x50	7,420	37762 T	31284 D	24046 D	18347 D	14216 D	10822 D	8118 D	6821 D	5812 D	5011 D	4365 D	3837 D	-	-	-	-	-	-	-	-	-
15x54.7	9,060	52052 T	41822 D	31857 D	24333 D	19065 D	14013 D	10755 D	9037 D	7700 D	6639 D	5784 D	5083 D	4503 D	4016 D	3605 D	-	-	-	-	-	-
18x70	9,060	69070 T	48822 D	37071 D	28559 D	19614 D	15078 D	12461 D	10471 D	8922 D	7693 D	6701 D	5890 D	5217 D	4654 D	4177 D	-	-	-	-	-	-
20x65.4	11,290	62569 T	47617 D	34864 D	26725 D	21163 D	17142 D	14167 D	11904 D	10143 D	8746 D	7619 D	6696 D	5932 D	5291 D	4749 D	4266 D	3887 D	-	-	-	-
20x75	11,290	74074 T	51440 D	37793 D	28535 D	22862 D	18519 D	15305 D	12860 D	10958 D	9448 D	8230 D	7234 D	6408 D	5716 D	5130 D	4630 D	4199 D	-	-	-	-
20x85	14,520	98738 T	73793 D	56968 D	45162 D	35684 D	28904 D	23888 D	20072 D	17103 D	14747 D	12846 D	11291 D	10001 D	8921 D	8007 D	7226 D	6554 D	-	-	-	-
20x95	14,520	92968 T	76137 D	63456 D	48584 D	38387 D	31094 D	25657 D	21593 D	18399 D	15864 D	13819 D	12146 D	10759 D	9597 D	8613 D	7773 D	7051 D	-	-	-	-
24x79.5	13,900	-	73511 D	54008 D	41250 D	32672 D	26464 D	21871 D	18376 D	15659 D	13502 D	11762 D	10338 D	9157 D	8168 D	7331 D	6616 D	6001 D	5465 D	5063 D	4515 D	4011 D
24x90	13,900	-	78029 D	57326 D	43892 D	34680 D	28091 D	23215 D	19507 D	16622 D	14352 D	12485 D	10973 D	9720 D	8670 D	7781 D	7023 D	6370 D	5804 D	5310 D	4835 D	4396 D
24x100	13,900	-	83969 D	61030 D	46726 D	36520 D	29905 D	24715 D	20767 D	17695 D	15258 D	13251 D	11682 D	10348 D	9230 D	8264 D	7476 D	6781 D	6179 D	5653 D	5172 D	4728 D
24x105.9	21,020	-	-	98143 D	76445 D	60400 D	48924 D	40433 D	33975 D	28948 D	24961 D	21744 D	19111 D	16929 D	15100 D	13552 D	12231 D	11094 D	10108 D	9248 D	8494 D	7828 D
24x120	21,020	-	-	-	82309 D	65034 D	52678 D	43535 D	36582 D	31170 D	26876 D	23412 D	20577 D	18228 D	16259 D	14592 D	13169 D	11945 D	10984 D	9958 D	9145 D	8428 D

Limits T = Tension C = Compression D = Vertical Deflection L = Lateral Deflection

* Recommended Lower Flange Loading per pair of wheels.

WIDE FLANGE SHAPES (W SHAPES)

WHC-SD SNE DA 016, Rev. 0

A41, pg 18 of 28

Section Designation Number	Depth of Section d in	Weight per Foot lb	Area of Section A in ²	Web Thickness t _w in	Flange Width b _f in	Flange Thickness t _f in	Area of Flange A _f in ²	d/A _x	Elastic Properties			Usual Flange Gage g in
									And I-X			
									I in ⁴	S in ³	r in	
W6x5	5.83	8.5	2.51	0.170	2.940	0.194	0.764	7.53	14.8	5.06	2.43	3 1/2
W6x10	6.00	15.5	4.56	0.235	5.995	0.269	1.612	3.72	30.1	10.0	2.57	3 1/2
W6x12	7.90	10.0	2.96	0.170	2.940	0.204	0.803	5.83	30.8	7.86	3.21	2 1/2
W6x17	8.00	17.0	5.01	0.230	5.250	0.308	1.617	4.95	56.6	14.1	3.36	2 1/2
W6x24	7.93	24.0	7.06	0.245	6.500	0.396	2.567	3.07	82.5	20.8	3.42	3 1/2
W6x31	8.00	31.0	9.12	0.286	6.000	0.433	3.464	2.51	110	27.4	3.47	5 1/2
W10x11.5	9.67	11.5	3.39	0.180	3.950	0.204	0.805	12.26	52	10.5	3.92	2 1/2
W10x21	9.90	21.0	6.20	0.240	5.750	0.340	1.955	5.06	107	21.5	4.15	2 1/2
W10x23	9.75	33.0	9.71	0.292	7.964	0.433	3.448	2.63	171	35.6	4.20	5 1/2
W10x49	10.00	49.0	14.4	0.340	10.000	0.558	5.580	1.79	273	54.6	4.35	5 1/2
W12x14	11.91	14.0	4.12	0.196	3.568	0.224	0.889	15.46	88	14.8	4.62	2 1/2
W12x27	11.96	27.0	7.95	0.237	6.467	0.400	2.596	4.86	204	34.2	5.07	3 1/2
W12x40	11.94	40.0	11.8	0.294	6.000	0.516	4.126	2.85	310	51.9	5.13	5 1/2
W12x53	12.06	53.0	15.6	0.345	10.000	0.576	5.760	2.09	426	70.7	5.23	5 1/2
W12x55	12.12	65.0	19.1	0.390	12.000	0.606	7.272	1.67	533	86.0	5.26	5 1/2
W14x22	13.72	22.0	6.46	0.230	5.000	0.335	1.675	8.19	198	26.9	5.52	2 1/2
W14x30	13.86	30.0	8.83	0.270	6.733	0.363	2.576	5.57	290	41.9	5.74	3 1/2
W14x43	13.66	43.0	12.6	0.308	8.000	0.528	4.224	3.24	429	62.7	5.82	5 1/2
W14x61	13.91	61.0	17.9	0.378	10.000	0.643	6.430	2.16	641	92.2	5.98	5 1/2
W14x76	14.06	76.0	22.3	0.428	12.000	0.718	8.616	1.63	851	121	6.09	5 1/2
W14x87	14.00	87.0	25.6	0.420	14.500	0.688	9.976	1.40	967	138	6.15	5 1/2
W16x26	15.65	26.0	7.67	0.250	5.500	0.345	1.897	8.25	300	36.3	6.25	2 1/2
W16x36	15.85	36.0	10.6	0.299	6.992	0.428	2.992	5.30	447	56.3	6.50	3 1/2
W16x56	15.86	56.0	17.1	0.407	8.464	0.645	5.459	2.91	748	94.4	6.62	5 1/2
W16x88	16.16	88.0	25.9	0.504	11.502	0.799	9.144	1.77	1220	151	6.67	5 1/2
W18x35	17.71	35.0	10.3	0.298	6.000	0.429	2.574	6.66	513	57.9	7.05	3 1/2
W18x45	17.86	45.0	13.2	0.335	7.477	0.499	3.731	4.79	706	79	7.30	3 1/2
W18x64	17.87	64.0	18.9	0.403	8.715	0.686	5.978	2.99	1050	118	7.46	5 1/2
W18x96	18.16	96.0	28.2	0.512	11.750	0.831	9.764	1.86	1680	185	7.70	5 1/2
W21x44	20.66	44.0	13.0	0.348	6.500	0.451	2.931	7.05	843	81.6	8.07	3 1/2
W21x55	20.80	55.0	16.2	0.375	8.215	0.522	4.288	4.85	1140	110	8.40	5 1/2
W21x82	20.86	82.0	24.2	0.499	1.962	0.795	7.124	2.53	1760	169	8.53	5 1/2
W21x112	21.00	112.0	33.0	0.527	13.000	0.865	11.245	1.57	2620	250	8.92	5 1/2
W24x55	23.55	55.0	16.2	0.396	7.000	0.503	3.521	6.69	1340	114	9.10	3 1/2
W24x68	23.71	68.0	20.0	0.416	1.961	0.562	5.215	4.59	1820	153	9.53	3 1/2
W24x76	23.91	76.0	22.4	0.440	8.985	0.602	6.126	3.90	2100	178	9.69	5 1/2
W24x100	24.00	100.0	29.5	0.468	12.000	0.775	9.300	2.56	3000	250	10.1	5 1/2
W27x84	26.69	84.0	24.8	0.463	9.963	0.636	6.336	4.21	2630	212	10.7	5 1/2
W27x94	26.91	94.0	27.7	0.490	9.990	0.747	7.462	3.61	3270	243	10.9	5 1/2
W30x99	29.64	99.0	29.1	0.522	10.458	0.670	7.006	4.23	4000	270	11.7	5 1/2
W30x108	29.82	108.0	31.8	0.548	15.484	0.760	7.968	3.75	4570	300	11.9	5 1/2
W33x118	32.86	118.0	34.8	0.554	11.484	0.738	8.475	3.86	5900	359	13.0	5 1/2
W36x135	35.55	135.0	39.8	0.598	11.945	0.794	9.484	3.75	7820	440	14.0	5 1/2
W36x150	35.84	150.0	44.2	0.625	11.972	0.940	11.254	3.16	9030	504	14.3	5 1/2
W36x230	35.68	230.0	67.7	0.761	16.471	1.260	20.753	1.73	15000	837	14.9	5 1/2

NOTE: Approximately 117 sizes of Wide Flange Shapes are produced in addition to those listed above. For information on these unlisted sizes, refer to "Manual of Steel Construction," 7th Edition.

Wide Flange Shapes have essentially parallel flange surfaces. Wide Flange Shapes produced by Jones & Laughlin Steel Corporation and Weston Steel Division of National Steel Corporation have a 3" taper in the inner flange surfaces. For exact information consult catalogs of these producers.

Wide Flange Shapes presently produced by Bethlehem Steel Corporation have parallel flange surfaces. Wide Flange Shapes produced by Bethlehem Steel Corporation prior to approximately 1967 had parallel flange surfaces with the following exceptions: All sizes with nominal

depths from 36 to 16 inches inclusive: W18x36 to 30; W12x36 to 27; W10x29 to 21; and W6x20 to 17. These Wide Flange Shapes had a 5 percent slope on the inner flange surfaces. For information on these shapes with the 5 percent slope, refer to "Bethlehem Structural Shapes," 1962 Edition.

Prior to approximately 1940, numerous sizes of Wide Flange Shapes were rolled in addition to those produced today. These beams are now considered obsolete, but may still be found in older installations. For such cases, where the obsolete shape designation is known or actual measurements of the beam indicate it is not one of the present standard shapes, refer to "Steel Construction," 3rd Edition, or contact the York office.

REFERENCES: See page 6

* Recommended Lower Flange Loading per pair of wheels.

W Section	Lower Flange Loading	Span in feet	
		10	12
6x8.5	375	1799 D	1222 D
6x15.5	765	3669 D	2495 D
6x18	870	2593 C	1775 C
6x17	1,000	6976 D	4790 D
6x24	1,680	10172 D	6986 D
6x31	1,990	13576 D	9375 D
10x11.5	670	2797 C	1913 C
10x21	1,230	12507 C	9140 D
10x33	1,990	20364 C	14610 D
10x49	3,310	31783 C	23753 D
12x14	670	3606 C	2470 C
12x27	1,700	18927 C	16556 C
12x40	2,830	30244 C	25130 C
12x53	3,540	41206 C	34242 C
12x65	3,910	51296 C	42627 C
14x22	1,500	11651 C	8035 C
14x30	1,560	24426 C	17861 C
14x43	2,870	36365 C	30357 C
14x61	4,400	53779 C	44704 C
14x78	5,500	70529 C	58681 C
14x97	6,450	80516 C	66937 C
16x26	2,040	15345 C	10590 C
16x36	1,950	32963 C	24473 C
16x55	4,430	55065 C	45795 C
16x88	6,740	88137 C	73286 C
18x25	2,040	27676 C	19270 C
18x45	2,650	46116 C	37532 C
18x64	5,020	68899 C	57298 C
18x96	7,370	-	69656 C
21x44	2,040	38368 C	26523 C
21x55	2,900	64251 C	52163 C
21x82	6,740	98725 C	82121 C
21x112	7,970	-	-
24x55	2,660	56339 C	39124 C
24x68	3,610	85610 C	74384 C
24x76	4,950	-	85576 C
24x100	6,410	-	-
27x84	6,300	-	-
27x94	5,950	-	-
30x99	4,780	-	-
30x108	6,160	-	-
33x118	5,710	-	-
36x135	6,710	-	-
36x150	9,420	-	-
36x230	16,900	-	-

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KAISER ENGINEERS
HANFORDCalc. No. ^{1/28} 21/A 20 of 28

Revision 0

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DESIGN ANALYSIS

client Westinghouse

WO/Job No. LE112

subject Alternate Analysis Check of Calcs. by
Harry Gotte of The Bartholomew Co. Inc.
for the 305 Bldg. Bridge Crane Mod.

Date 9/20/95

By Mark Axup

Checked 9/27/95

By *Mark Axup*

Location 305 Building

Revised

By

Note: Section numbers shown correspond to section numbers on original calculation by Harry Gotte (H. G.) of The Bartholomew Co.

2. Bridge girder (W14x61, ref. dwg. H-4-101657, sht. 1 of 3)

Design loading and stress analysis for 2 ton bridge girder hoist & trolley.

Design Loading:

Vertical: Lifted Live Load = 4000.0 #
 hoist & Trolley = $\frac{1145.0}{5145.0}$ # - See Section 1.0 of H.G. Calc.
 10 % Impact $\frac{514.5}{5659.5}$ # - Use Section A4.2

$$\text{wheel load} = \frac{5659.5}{4} = 1414.9 \text{ # say } \underline{1415.0 \text{ #}}$$

Lateral

$$LL - \text{hoist \& trolley} = 5145.0$$

$$\text{wheel load} = \frac{5145.0}{4} = 1286.3 \text{ #}$$

$$\text{lateral force/wheel} = 1286.3 (0.20) = 257.3 \text{ #}$$

say 260.0 #

Longitudinal

$$\text{max. load/girder} = 2 (1415.0) = 2830.0 \text{ #}$$

$$\text{long. force} = 2830.0 (0.10) = \underline{283.0 \text{ #}}$$

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

client: Westinghouse
 subject: Alternate Analysis Check of Calcs. by
 Harry Gotte of The Bartholomew Co. Inc.
 for the 305 Bldg. Bridge Crane Mod.
 Location: 305 Building

WO/Job No. LE112

Date 9/20/95

By Mark Axup

Checked 9/27/95

By *Wayne Clark*

Revised 12/7/95

By *Wayne Clark*

2. Bridge girder (cont'd.)

Stress Analysis:

max. moments & stress: see loading condition 41 disc manual, p. 2-310

$a = 3.17'$ - see dwg. H-4-101457 slt. 1 of 3

$L = 31.0'$ - see dwg. H-4-101454

$3.17 < 0.58L = 18.17' \therefore M_{max}$ occurs under load

$$x = \frac{1}{2} \left[31.0 - \frac{3.17}{2} \right] = 14.71'$$

Vertical

$$R_R = \left[0.070 (31.0)^2 / 2 + 1.415 (14.71 + 17.85) \right] / 31.0 = 2.573 \text{ k}$$

$$R_L = 0.070 (31.0) + 2(1.415) - 2.573 = 2.427 \text{ k}$$

$$M_{x_{max}} = 2.427 (14.71) - 0.070 (14.71)^2 / 2 = 28.13 \text{ k-ft}$$

$$R_{max} = 1.415 \left[2 - \frac{3.17}{31.0} \right] + \frac{0.070 (31.0)}{2} = 3.770 \text{ k}$$

(by inspection)

KAISER ENGINEERS
HANFORD

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DESIGN ANALYSIS

client Westinghouse
 Subject Alternate Analysis Check of Calcs. by
 Harry Gotte of The Bartholomew Co. Inc.
 for the 305 Bldg. Bridge Crane Mod.
 Location 305 Building

WO/Job No. LE112

Date 9/20/95

Checked 9/27/95

Revised

By Mark Axup

By *Mark Axup*

By

2. Bridge girder (cont'd.)

Stress Analysis (cont'd.)Lateral

$$W_{Tmax} = \frac{0.260}{2(31.0)} \left[31.0 - \frac{3.17}{31.0} \right]^2 = 4.00 \text{ in}^2$$

W14x61 Section Properties:

$d = 17.9 \text{ in}^2$	$F_y = 2.70 \text{ in}$	$L_c = 10.6'$
$d = 13.69 \text{ in}$	$d/A_f = 2.15 \text{ in}$	$L_u = 21.5'$
$t_w = 0.375 \text{ in}$	$S_x = 92.2 \text{ in}^3$	$S_y =$
$b_w = 9.995 \text{ in}$	$T_x = 5.98 \text{ in}$	(top half of beam only)
$t_f = 0.645 \text{ in}$	$I_y = 107.0 \text{ in}^4$	$I_y/2 = \frac{107.0/2}{9.995/2} = 10.71 \text{ in}^2$
$I_x = 640.0 \text{ in}^4$	$I_y = 2.45 \text{ in}^4$	

$$L/r_z = \frac{31.0(12)}{2.70} = 137.8 > \sqrt{\frac{510 \times 10^3 C_b}{F_y}} = 119.02$$

$$F_{bx} = \frac{170 \times 10^3 C_b}{(L/r_z)^2} = \frac{170 \times 10^3 (1.0)}{(137.8)^2} = 8.95 \text{ ksi}$$

$$F_{bx} = \frac{12 \times 10^3 C_b}{d_d/d_f} = \frac{12 \times 10^3 (1.0)}{12(31.0)(2.15)} = 15.00 \text{ ksi} \quad \leftarrow \text{Use}$$

DESIGN ANALYSIS

Client Westinghouse

MO/Job No. LE112

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Date 9/20/95

By Mark Axup

Checked 9/27/95

By *Wayne Adair*

Location 305 Building

Revised 12/7/95

By *Wayne Adair*

2. Bridge girder (cont'd.)

Stress Analysis: (Cont'd.)

Top Flange Stress $f_{bx} = \frac{26.13 (12)}{92.2} = 3.361 \text{ ksi}$

$f_{by} = \frac{4.00 (12)}{10.71} = 4.482 \text{ ksi}$

$\frac{f}{\text{long.}} = \frac{0.283}{(1.995)(0.645)} = 0.044 \text{ ksi}$

$f_{\text{total}} = 5.187 \text{ ksi} < 15.0 \text{ ksi OK}$

bottom flange stress OK by inspection

check deflection: Conservatively, assume concentrated load of
 $2 \times 1.45 = 2.83 \text{ K}$ acts @ center

$\Delta = \left[\frac{5(0.070) 31.0^4}{384 (29 \times 10^3) 640} + \frac{2.83 (31.0)^3}{48 (29 \times 10^3) 640} \right] 1729 = 0.2419" = L/1538 \text{ OK}$

DESIGN ANALYSIS

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WO/Job No. LE112

Date 9/20/95

By Mark Axup

Checked 9/27/95

By *Wagner*

Revised

By

6. Monorail Analysis: 2 ton capacity

Use Reduced Allowable Stress Method AWS 27.1-1957

Weight of hoist & trolley = 411.0 # - see page 4 of H.G. calcs.

Lift = 4000.0 #

Impact = 15% of Lift = $\frac{600.0}{5011.0}$ # - see Section 2.3.4

Allowable stresses: $\bar{F}_b = 0.2 (58.0) = 11.6$ ksi Section 3.1.1
Tension (Controls)

Try 5×18.4
section Properties:

$S_x = 14.4$ in³

$I_x = 57.6$ in⁴

$r_x = 0.94$ in

$d/A_f = 4.70$

$$L/r_x = \frac{10.0 (12)}{0.94} = 127.66$$

$$\bar{F}_{bx} = \frac{170 \times 10^3 (1.0)}{(127.66)^2} = 10.43$$

$$\bar{F}_{by} = \frac{12 \times 10^3 (1.0)}{10.0 (12) (4.70)} = 21.28$$

$$M_{max} @ ch. = \frac{5.011 (10.0)}{4} + \frac{0.02 (10.0)^2}{8} = 12.78 \text{ ft-k}$$

$$\text{check stress: } \frac{M}{S_x} = \frac{12.78 (12)}{14.4} = 10.65 \text{ ksi} < 11.6 \text{ ksi OK}$$

check deflection:

$$\Delta = \left[\frac{5.011 (10.0)^3}{48 (29 \times 10^3) 57.6} + \frac{5 (0.02) (10.0)^3}{384 (29 \times 10^3) 57.6} \right] 1728 = 0.110" \approx L/1094 \text{ OK}$$

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client Westinghouse
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 for the 305 Bldg. Bridge Crane Mod.
 Location 305 Building

WD/Job No. LE112

Date 9/20/95 By Mark Axup

Checked 9/27/95

By Wayne Adell

Revised

By

8. Bridge Girder Analysis for Underhung Monorail System - New slave girder only (with rail)

Design Loading:

Vertical: Conservatively use 2882 #/wheel - see Section 7.0 of H.G. Calc.

Lateral L.L. + hoist trolley 4411 # - see Section 6.0 of H.G. Calc.

$$\begin{array}{r}
 111 \\
 30 \\
 288 \\
 323 \\
 \hline
 5163 \text{ # Total}
 \end{array}
 \left. \begin{array}{l} \\ \\ \\ \end{array} \right\} \text{ Misc, see Section 7.0 of H.G. Calc.}$$

lateral force/wheel = $\frac{5163}{2} (0.20) = 516.3$ #

Assume only slave girder resists $\rightarrow$ Disc Section A4.3

lateral load $\therefore$ divide by 2

instead of 4

say 520.0 #

Longitudinal max. load/girder = $2(2882) = 5764$ #

longitudinal force = $5764.0 (0.10) = 576.4$ #

Disc Section A4.3

Stress Analysis Max. Moments & Shears - see loading condition 41
 Disc Manual, p. 2-3.10

a. 5.0' - see Section 3.0 H.G. Calc.

l. 31.0' - see diag. H-4-101654

$5.0' < 0.576 \times 18.17'$ $\therefore$ Max occurs under load

@ $x = \frac{1}{2} \left[31.0 - \frac{5.0}{2} \right] = 14.25'$

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Revision 1

Page No. 7 of

DESIGN ANALYSIS

client Westinghouse
 subject Alternate Analysis Check of Calcs. by
 Harry Gotte of The Bartholomew Co. Inc.
 for the 305 Bldg. Bridge Crane Mod.
 Location 305 Building

WO/Job No. LE112

Date 9/20/95

Checked 9/27/95

Revised 12/7/95

By Mark Axup

By *Wayne Adams*By *Wayne Adams*

8. Bridge Girder Analysis for Underhung Monorail System - New slave girder only (without)

Stress Analysis: (Cont'd.)

Vertical

$$R_R = [0.070 (31.0)^2 / 2 + 2.882 (14.25 + 19.25)] / 31.0 = 4.199 \text{ K}$$

$$R_L = 0.070 (31.0) + 2.882 (2) - 4.199 = 3.735 \text{ K}$$

$$M_{\text{max}} = 3.735 (14.25) - 0.070 (14.25)^2 / 2 = 46.12 \text{ K-ft}$$

$$R_{\text{max}} = 2.882 \left[2 - \frac{5.0}{31.0} \right] + \frac{0.070 (31.0)}{2} = 6.384 \text{ K}$$

(by inspection)

Lateral

$$M_{\text{ly max}} = \frac{0.520}{2 (31.0)} \left[31.0 - \frac{5.0}{2} \right]^2 = 6.81 \text{ K-ft}$$

top flange stress: $f_{\text{top}} = \frac{46.12 (12)}{92.2} = 6.003 \text{ ksi} < 15.0 \text{ ksi OK}$

bottom flange stress: $f_{\text{bot}} = \frac{46.12 (12)}{92.2} = 6.003 \text{ ksi}$

$f_{\text{bot}} = \frac{6.81 (12)}{10.71} = 7.630 \text{ ksi}$

$\frac{1}{\lambda} = \frac{0.5764}{(9.915)(0.645)} = 0.089 \text{ ksi say negligible}$

 see page 3 of
 alt. calcs.

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Checked 9/27/95

By *Wayne Hall*

Revised 12/7/95

By *Wayne Hall*

8. Bridge Girder Analysis for Underhung Monorail System

Stress Analysis : (Cont'd.)

bottom flange stress : check combined stress

(Cont'd.)

$$f_b = 6.003 + 7.630 = 13.633 \text{ ksi}$$

Tension

Per AISC F1.2, eq. (F1-5)

$$F_u = 0.6(36.0) = 21.6 \text{ ksi} > 13.633 \text{ ksi} \quad \text{OK}$$

Tension

10. Simultaneous Girder Loads (w/under)

Load case with underhung 2 ton monorail located directly under new column attached to existing bridge girder. Both monorail and 2 ton bridge hoist loaded to capacity.

Assumed that simultaneous lateral impact by the bridge hoist and the underhung monorail system is unlikely. Only lateral impact from the bridge crane hoist will be accounted for.

Use stresses previously calculated but subtract stress due to uniform load of 0.100 k/ft so that it is not accounted for twice

$$M_u = \frac{0.070(31.0)^2}{8} = 8.409 \text{ k-ft} \quad f_b = \frac{8.409(12)}{92.2} = 1.094 \text{ ksi}$$

uniform load

Stresses due to combined loading :

$$\begin{array}{l} \text{Top Flange Stress} \quad \frac{P}{A} = 8.187 + 6.003 - 1.094 = 13.096 \text{ ksi} < 15.0 \text{ ksi} \quad \text{OK} \\ \text{(compression)} \quad \text{Total} \end{array}$$

$$\begin{array}{l} \text{bottom flange stress} \quad \text{OK by inspection} \\ \text{(tension)} \end{array}$$

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WO/Job No. LE112
Date 9/22/95 By Mark Axup
Checked 9/27/95 By Wayne Adair
Revised 12/17/95 By Wayne Adair

15. Original Girder Connection

Check comp. section for shear.

max. Design Shear: $\rightarrow$ p. 7 of this alternate check calculation

$$V_{max} = 3.770 + 6.384 = 10.154 \text{ k}$$

$\leftarrow$ p. 2 of this
alternate check
calculation

Note: This value is very conservative
since it includes twice the
beam uniform load and
was derived assuming the
load was applied directly over
the end of the beam.

Check Shear Stress

Remaining beam depth = 7.375"

$$t_w = 0.645$$

$\rightarrow$ Ref. Disc, Section F4.

$$\frac{V}{t_w} = \frac{10.154}{(0.645)(7.375)} = 2.135 \text{ ksi} < 0.4(36.0) = 14.4 \text{ ksi} \text{ OK}$$

Check Local Web Yielding

Length of bearing = 12.5"

$\rightarrow$ Ref. Disc, Section K1.3

$$f_a = \frac{10.154}{(0.645)(12.5)} = 1.259 \text{ ksi} < 0.66(36.0) = 23.76 \text{ ksi} \text{ OK}$$

(along web)

16. NEW GIRDER CONNECTION:

$$V_{max} = 6.384 \text{ k} \text{ (see above)}$$

Check Shear Stress:

Remaining beam depth = 27/8"

$$\frac{V}{t_w} = \frac{6.384}{(2.875)(0.645)} = 3.44 \text{ ksi} < 14.4 \text{ ksi} \text{ OK}$$

Local web yielding OK by inspection

Attachment 2:

Letter for 305 Building Crane Structure Analysis

by

Gary Wagenblast, WHC

From: Facilities Engineering
Phone: 6-0171
Date: August 23, 1983
Subject: 305 CRANE STRUCTURAL ANALYSIS

Westinghouse Hanford Company

To: C. R. Allen

cc: SA Dawson W/A-124
RC Ingersoll W/B-108
GRW/LB

Reference: Letter, C. R. Allen to R. C. Ingersoll, same subject,
dated July 27, 1983.

The 305 building crane structure depicted on drawings H-3-34291 and H-3-34292 was analyzed as requested in the reference letter.

The upper rails have the capacity to withstand one bridge crane having a capacity of 10 tons or two bridge cranes having a 5 ton capacity each. The combination of one 10 ton bridge and one 5 ton bridge, as proposed, would require modifications to the rail beams and modifications to all beam-to-column attachments.

The lower rails are adequate for the existing electromechanical manipulator bridge. The addition of a second manipulator bridge would require either a modification to the rail beams or physical barriers to assure a separation of 5 feet between bridges. No modifications to beam attachments would be required in either case.

Gary Wagenblast
G. R. Wagenblast
Engineer

ma

DISTRIBUTION SHEET

To Spent Nuclear Fuels Engineering	From M. D. Axup Analytical Services	Page 1 of 1 Date January 23, 1996
Project Title/Work Order 305 BUILDING 2 TON BRIDGE CRANE AND MONORAIL ASSEMBLY ANALYSIS		EDT No. 614408 ECN No.

Name	MSIN	Text With All Attach.	Text Only	Attach./ Appendix Only	EDT/ECN Only
M. D. Axup	S2-03	X			
S. K. Farnworth	H5-57				X
J. R. Frederickson	R3-86				X
L. H. Goldman	R3-86	X			
T. E. Griffin	L6-13	X			
T. C. Mackey	S2-03	X			
M. J. Schliebe	L6-13				X

Central Files

A3-88 (2)