# 11-4403-cv

# United States Court of Appeals

for the

# Second Circuit

AEGIS INSURANCE SERVICES, INC., LIBERTY INSURANCE UNDERWRITERS, INC., NATIONAL UNION INSURANCE COMPANY OF PITTSBURGH, NUCLEAR ELECTRIC INSURANCE LIMITED, CERTAIN UNDERWRITERS AT LLOYDS, (Syndicates 1225 and 1511), as subrogor of Consolidated Edison Company of New York, Inc., and CONSOLIDATED EDISON COMPANY OF NEW YORK, INC.

Plaintiffs-Appellants,

(For Continuation of Caption See Inside Cover)

ON APPEAL FROM THE UNITED STATES DISTRICT COURT FOR THE SOUTHERN DISTRICT OF NEW YORK

## JOINT APPENDIX Volume 15 of 16 (Pages JA-3887 to JA-4186)

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(For Continuation of Appearances See Inside Cover)

v.

## 7 WORLD TRADE COMPANY, L.P.,

Defendant-Cross-Defendant-Cross-Claimant-Third-Party Plaintiff-Appellee,

CITIGROUP INC., CITIGROUP GLOBAL MARKETS HOLDINGS INC., SALOMON SMITH BARNEY HOLDINGS, INC., SALOMON INC., SILVERSTEIN DEVELOPMENT CORP., SILVERSTEIN PROPERTIES, INC.,

Defendants-Cross-Defendants-Cross-Claimants-Appellees,

## TISHMAN CONSTRUCTION CORPORATION.

Defendant-Cross-Defendant-Appellee,

OFFICE OF IRWIN G. CANTOR, P.C., FLACK & KURTZ, INC.,

Defendants-Cross-Defendants-Third-Party Defendants-Appellees,

SWANKE HAYDEN CONNELL ARCHITECTS, SYSKA & HENNESSY GROUP, INC., AKA SYSKA & HENNESSY ENGINEERS,

Defendants-Cross-Defendants-Cross-Claimants-Third-Party Plaintiffs,

H.O. PENN MACHINERY CO., INC., ALL FIRE SYSTEMS, INC.,

Defendants-Cross-Defendants-Cross-Claimants,

GRACE CONSTRUCTION PRODUCTS, EMERY ROTH & SON, P.C., SECURITAS AB, SECURITY SERVICES, INC., CENTIFUGAL ASSOCIATES, INC., SYSKA & HENNESSY, INC.,

Defendants-Cross-Defendants,

AMBASSADOR CONSTRUCTION CO., INC., COSENTINI ASSOCIATES INC., CANTOR SEINUK GROUP, P.C., SKIDMORE OWINGS AND MERRILL, L.L.P., AMBASSADOR CONSTRUCTION CO., INC., AMEC CONSTRUCTION MANAGEMENT, INC., FKA MORSE DIESEL INTERNATIONAL, INC.,

Defendants-Cross-Defendants-Third-Party Defendants,

DIC/UNDERHILL, a joint venture, KABACK ENTERPRISES, PREFERRED UTILITIES MANUFACTURING CORP., ELECTRIC POWER SYSTEMS, INC., G.C. ENGINEERING & ASSOCIATES, P.C., FIRECOM INC., FIBERLOCK TECHNOLOGIES, INC., ROSEBWACH TANK CO., INC., ABCO PEERLESS SPRINKLER CORPORATION, AMR CORPORATION, AMERICAN AIRLINES, INCORPORATED, UAL CORPORATION, UNITED AIRLINES INCORPORATED, COLGAN AIR, INCORPORATED, US AIRWAYS GROUP, INCORPORATED, US AIRWAYS, INCORPORATED, HUNTLEIGH USA CORPORATION, ICTS INTERNATIONAL NV, GLOBE

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AVIATION SERVICES CORPORATION, BURNS INTERNATIONAL SECURITY SERVICES CORPORATION, PINKERTON'S INCORPORATED, BOEING COMPANY, AMEC, PLC, KABACK ENTERPRISES,

Defendants,

## THE WTC PLAINTIFFS,

Plaintiff-Intervenor.

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Exhibit 32: Deposition Exhibit Glass-8, which was Marked as an Exhibit at John Glass' Deposition, Dated November 11, 2008	JA-1855
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Exhibit 57: Excerpts of the Transcript of the Deposition Testimony of Mark A. Giannini, Dated June 29, 2009	JA-1985
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Exhibit 68: Second Amended Complaint, Filed July 11, 2008 (Omitted Here but Reproduced at p. JA-704)	JA-2026
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Exhibit 71: Affidavit of Barbara A. Carey, Dated October 27, 2008	JA-2029
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Exhibit 78: U.S. Court of Appeals for the Second Circuit, General Docket Referencing the Current Appellate Scheduling Order	JA-2054
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Second Declaration of Colin G. Bailey, Filed February 1, 2010, with the Following Attached Exhibits:	JA-2084
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Second Declaration of Joseph P. Colaco, Filed February 1, 2010, with the Following Attached Exhibit	JA-2110
Exhibit A: Joseph P. Colaco's Curriculum Vitae	JA-2114
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Second Declaration of Guy Nordenson, Filed February 1, 2010, with the Following Attached Exhibit	JA-2155
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Exhibit A: Jose L. Torero's Curriculum Vitae	JA-2178
Plaintiffs' Response to Defendants 7 World Trade Company, L.P., Silverstein Development Corp. and Silverstein Properties, Inc. Local Civil Rule 56.1 Statement in Support of Defendants' Motion for Summary Judgment Dismissing Plaintiffs' Claims, Filed February 1, 2010	JA-2206
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Declaration of Sheela V. Pai, Esq. in Support of the Motion for Summary Judgment, Filed March 19, 2010, with the Following Attached Exhibits	JA-2249
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Exhibit E: Excerpts of the Transcript of the Deposition Testimony of Peter Joseph Mulroy, Dated August 27, 2007	JA-2269
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Exhibit J: Excerpts of the Transcript of the Deposition Testimony of Anthony Zeolla, Dated June 18, 2009	JA-2282
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Exhibit M: Excerpts of Report Entitled: "Response to WTC7 to Standard Office Fires and Collapse Initiation," Authored by Professor Colin Bailey, Dated February 15, 2010	JA-2288
Exhibit N: Excerpts of Report and Summary of Findings:  "Global Collapse Analysis – World Trade Center 7  Collapse Investigation," Authored by Guy Nordenson,  Dated February 12, 2010	JA-2290
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Exhibit P: Excerpts of the Transcript of the Deposition Testimony of Chi Chu, Dated November 20, 2008	JA-2296
Exhibit Q: Excerpts of the Transcript of the Deposition Testimony of Richard A. Rotanz, Dated May 14, 2009	JA-2298
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Exhibit S: Excerpts of the Transcript of the Deposition Testimony of Joseph Ritorto, Dated August 7, 2008	JA-2303
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Exhibit 4: Excerpts of the Transcript of the Deposition Testimony of Eugene Fasullo, Dated April 11, 2008(Further Supplementing Santillo Declaration Exhibit 14, Dated December 18, 2007)	JA-2490
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Exhibit 8: Excerpts of the Transcript of the Deposition Testimony of Rudolph C. Weindler, Dated May 6, 2009 (Further Supplementing Santillo Declaration Exhibit 27).	JA-2503
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Exhibit 12: Excerpts of the Transcript of the Deposition Testimony of Peter Hayden, Dated May 20, 2009 (Further Supplementing Santillo Declaration Exhibit 32).	JA-2518
Exhibit 13: Excerpts of the Transcript of the Deposition Testimony of John Spiech, Dated May 27, 2009 (Further Supplementing Santillo Declaration Exhibit 34)	JA-2521

Exhibit 14: Con Edison September 11, 2001 Restoration Costs  — Summary of 7 World Trade Center Damage Claim  Spreadsheet, Dated May 31, 2009	JA-2524
Exhibit 15: Excerpts of the Transcript of the Deposition Testimony of Anthony Varriale, Dated June 9, 2009 (Further Supplementing Santillo Declaration Exhibit 36).	JA-2526
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Plaintiffs' Notice of Motion for Leave to File Supplemental and Amended Expert Declarations Incorporating Plaintiffs' Expert Reports Pursuant to Fed. R. Civ. P. 56(e)(1) in Further Opposition to the Motions for Summary Judgment of Silverstein and Citigroup, Filed April 6, 2010, with the Following Attachments	JA-2549
Supplemental and Amended Second Declaration of Colin G.     Bailey, Dated April 5, 2010, with Exhibits	JA-2552
2. Supplemental and Amended Second Declaration of Joseph P. Colaco, Dated April 1, 2010, with Exhibits	JA-2584
3. Supplemental and Amended Second Declaration of Kenneth Elovitz, Dated April 1, 2010, with Exhibits	JA-2600
4. Supplemental and Amended Second Declaration of Frederick W. Mowrer, Dated April 1, 2010, with Exhibits	JA-2613

5. Supplemental and Amended Second Declaration of Guy Nordenson, Dated April 1, 2010, with Exhibits	JA-2639
6. Supplemental and Amended Second Declaration of Joseph M. Sorge, Dated April 1, 2010, with Exhibit	JA-2655
7. Supplemental and Amended Declaration of Jose L. Torero, Dated April 1, 2010, with Exhibit	JA-2664
Declaration of Franklin M. Sachs, Esq. in Support of Plaintiffs' Motion for Leave to File Supplemental and Amended Expert Declarations Incorporating Plaintiffs' Expert Reports Pursuant to Fed. R. Civ. P. 56(e)(1) in Further Opposition to the Motions for Summary Judgment of Silverstein and Citigroup, Filed April 6, 2010, with the Following Attached Exhibits	JA-2698
Exhibit 1: Letter from Franklin M. Sachs, Esq. to Katherine L. Pringle, Esq. and Christopher P. Moore, Esq., Dated February 16, 2010	JA-2702
Exhibit 2: Reply Brief in Support of Motion by Defendants 7 World Trade Company, L.P., Silverstein Development Corp. and Silverstein Properties Inc. for Summary Judgment Dismissing All Claims, Dated March 19, 2010.	JA-2704
Exhibit 3: Declaration of Sheela V. Pai, Esq. in Support of Motion for Summary Judgment, Dated March 19, 2010, with Exhibits	JA-2713
Exhibit 4: Declaration of Craig L. Beyler, Dated March 18, 2010, with Exhibits	JA-2736
Exhibit 5: Citigroup Inc. and Citigroup Global Markets Holdings, Inc.'s Reply Memorandum of Law in Support of the Motion for Summary Judgment, Dated March 19, 2010	JA-2773
Exhibit 6: Declaration of Megan M. St. Ledger, Esq. in Further Support of Citigroup Inc. and Citigroup Global Markets Holdings Inc.'s Memorandum of Law in Support of the Motion for Summary Judgment, Filed March 19, 2010,	
with Exhibits	JA-2784

Exhibit 7: Declaration of Chester T. Vogel, Dated March 19, 2010, with Exhibits	JA-2812
Exhibit 8: Citigroup Inc. and Citigroup Global Markets Holdings Inc.'s Reply Statement of Undisputed Material Facts, Filed March 19, 2010 (Omitted Here but Reproduced at p. JA-2344)	JA-2847
Declaration of Katherine L. Pringle, Esq. in Support of the Accompanying Opposition by Defendants 7 World Trade Company, L.P., Silverstein Development Corp. and Silverstein Properties Inc. to Plaintiffs' Motion for Leave to File Supplemental and Amended Expert Declarations, Filed April 21, 2010, with the Following Attached Exhibits	JA-2848
Exhibit A: Letter from Katherine L. Pringle, Esq. to the Honorable Alvin K. Hellerstein, Dated November 12, 2009	JA-2850
Exhibit B: Second Declaration of Kenneth Elovitz, Dated January 22, 2010	JA-2852
Exhibit C: Excerpts of Citigroup Inc. and Citigroup Global Markets Holdings, Inc.'s Reply Memorandum of Law in Support of the Motion for Summary Judgment, Filed March 19, 2010	JA-2864
Supplemental and Amended Declaration of Jose L. Torero, Dated April 1, 2010, with the Following Attached Exhibits	JA-2866
Exhibit A: Jose L. Terero's Curriculum Vitae	JA-2869
Exhibit B: Report Entitled "Analysis of the Impact of a Fire in the Mechanical Room (5 <sup>th</sup> & 6 <sup>th</sup> Floor) of the World Trade Center 7 Building," Dated February 12, 2010	JA-2897
Supplemental and Amended Second Declaration of Joseph P. Colaco, Dated April 1, 2010, with the Following Attached Exhibits	JA-2979
Exhibit A: Joseph P. Colaco's Curriculum Vitae	JA-2983
Exhibit B: CBM Engineers, Inc., World Trade Center 7: Joseph Colaco Expert Report, Dated February 15, 2010	JA-2993

Supplemental and Amended Second Declarations of Colin G. Bailey, Dated April 5 <sup>th</sup> and 10 <sup>th</sup> , 2010, with the Following Attached	
Exhibits	JA-3067
Exhibit 1: Colin G. Bailey's Curriculum Vitae	JA-3075
Exhibit A: Photographs Showing Fire Protection on a Different Building	JA-3086
Exhibit B: Photograph of SFRM Being Applied	JA-3096
Exhibit C: Photograph of Flutes Filled with SFRM	JA-3098
Exhibit D: Response of WTC7 to Standard Office Fires and Collapse Initiation, Dated February 15, 2010, By Professor Colin Bailey	JA-3100
Supplemental and Amended Second Declaration of Frederick W. Mowrer, Filed April 22, 2010, with the Following Attached Exhibits	JA-3299
Exhibit A: Frederick W. Mowrer's Curriculum Vitae	JA-3305
Exhibit B: Photograph of the East Side of 7 World Trade Center	JA-3315
Exhibit C: Morse Diesel Photographs Depicting Unfilled Flute Cavities	JA-3316
Exhibit D: Expert Report by Frederick W. Mowrer, Ph.D., Dated February 15, 2010	JA-3320
Supplemental and Amended Second Declaration of Guy Nordenson, Filed April 22, 2010, with the Following Attached Exhibits	JA-3887
Exhibit A: Guy JP Nordenson's Curriculum Vitae	JA-3890
Exhibit B: Report and Summary of Findings: Global Collapse Analysis: World Trade Center 7, Collapse Investigation, Dated February 12, 2010	JA-3900
Supplemental and Amended Second Declaration of Kenneth Elovitz, Filed April 22, 2010, with the Following Attached Exhibits	JA-4229
Exhibit A: Kenneth M. Elovitz's Curriculum Vitae	JA-4234

Exhibit B: Expert Report of Energy Economics, Inc., Dated February 11, 2010	JA-4239
Supplemental and Amended Second Declaration of Joseph M. Sorge and Curriculum Vitae, Filed April 22, 2010, with the Following Attached Exhibit	JA-4258
Exhibit A: JM Sorge, Inc.'s Expert Report, Dated February 2010	JA-4265
Plaintiffs' Memo Endorsed Notice of Motion for Leave to File Supplemental and Amended Expert Declarations Incorporating Plaintiffs' Expert Reports Pursuant to Fed. R. Civ. P. 56(e)(1) in Further Opposition to the Motions for Summary Judgment of Silverstein and Citigroup, Signed by Judge Alvin K. Hellerstein on April 23, 2010.	JA-4297
Memo Endorsed Letter from Franklin M. Sachs, Esq. to the Honorable Alvin K. Hellerstein, U.S.D.J., Dated November 19, 2010	JA-4300
Transcript of Argument, Dated December 16, 2010	JA-4304
Memo Endorsed Letter from Franklin M. Sachs, Esq. to the Honorable Alvin K. Hellerstein, U.S.D.J., Dated March 16, 2011	JA-4390
Stipulation of Voluntary Dismissal of Cross-Claims, Filed November 3, 2011	JA-4393
Stipulation of Voluntary Dismissal of Cross-Claims, Filed November 3, 2011	JA-4395
Memo Endorsed Letter from Katherine L. Pringle, Esq. to the Honorable Alvin K. Hellerstein, Dated November 3, 2011	JA-4397
Defendant Tishman Construction Corporation's Master Answer to Property Plaintiffs' Master Complaint Against Ground Defendants, Dated December 1, 2005	JA-4399
Transcript of Court Conference, Dated June 21, 2005	JA-4414
Transcript of Conference, Dated August 9, 2006	JA-4463

Memo Endorsed Letter, Dated August 23, 2010, from Beth D. Jacob,	
Esq. to The Honorable Alvin K. Hellerstein, Requesting	
Removal of Exhibit 17 to the Declaration of Jemi M. Goulian,	
Esq., Filed February 1, 2010, from the Public File, Signed by	
Judge Hellerstein on August 24, 2010	JA-4481

Franklin M. Sachs (FS6036) GREENBAUM, ROWE, SMITH & DAVIS LLP Metro Corporate Campus One P.O. Box 5600 Woodbridge, New Jersey 07095 Telephone: (732) 549-5600

# UNITED STATES DISTRICT COURT SOUTHERN DISTRICT OF NEW YORK

	x
IN RE: SEPTEMBER 11 PROPERTY DAMAGE AND BUSINESS LOSS LITIGATION	: 21 MC 101 (AKH)
AEGIS INSURANCE SERVICES, INC., et al.,	: 04 CV 7272 (AKH)
Plaintiffs,	1
-against-	: SUPPLEMENTAL AND
	: AMENDED SECOND
7 WORLD TRADE CENTER COMPANY, L.P.,	DECLARATION OF
et al.,	GUY NORDENSON
Defendants.	
	1
	X

#### I, Guy Nordenson, declare:

- I am a professor of architecture and structural engineering at Princeton University and a
  practicing structural engineer in New York City. I am a licensed Civil and Structural
  Engineer in California and a licensed Professional Engineer in New York State as well as
  other states. Among my specialties are tall building structural design, earthquake
  engineering and the analysis and design of special structures. My curriculum vitae is
  attached hereto as Exhibit A.
- 2. In 2007, I was retained by counsel for plaintiffs in this litigation to serve as consulting structural engineer. I make this affidavit based upon the work that I have done in studying the possible effects of the local failure of a structural member or connection on the total collapse of 7 World Trade Center (WTC7).
- 3. Since that time, I have reviewed thousands of documents, drawings, and photographs, I have reviewed the computer fire modeling performed on behalf of the Plaintiffs in this

- case and I have performed computer structural analyses upon which my opinions regarding the cause of the global collapse of the building are based.
- 4. The opinions that follow are based on that review and activity, and are made to a reasonable degree of scientific probability. These opinions and the data and materials relied upon in forming these opinions are more fully set forth in my report dated February 12, 2010, attached hereto as Exhibit B and made a part hereof.
- Based upon my review of available photographic and video evidence, and the deposition testimony of eyewitnesses, including members of the F.D.N.Y., it is my opinion that the collapse of WTC1 or WTC2 did not cause structural damage to any of the core columns of WTC7.
- 6. The perimeter moment frame columns and the core columns of WTC7 are different in kind. WTC7, prior to its collapse, had 58 perimeter columns that were rigidly connected to spandrel beams to form a moment frame. The interior core columns were not rigidly connected to the perimeter moment frame. Therefore the loss of six or seven perimeter columns in the southwest corner and/or the south side of the building would not have contributed to the collapse of the entire building.
- 7. Based upon the work performed by Colin G. Bailey, which I have reviewed, the failure to adequately fireproof the flutes of the metal decking of WTC7, and the failure to ensure that a restrained floor system was constructed, would have initiated the collapse sequence of WTC7 from an ordinary office contents fire, along the column line of Columns 79, 80, and 81, likely at Column 79, between the ninth and thirteenth floors.
- 8. Because of the very large open floor bays supported by Column 79, a local floor failure near Column 79 between the ninth and thirteen floors would lead to a collapse of the floors adjacent Column 79, at least to the fifth floor, if not all the way to the ground. That collapse would destabilize Column 79 and then Column 80 as a result of their inadequate lateral bracing. This behavior was evident by the sinking of the east penthouse below the roofline along the column line of Columns 79, 80 and 81.
- 9. Based upon the work performed by Jose L. Torero, which I have reviewed, a fire caused by the ignition of diesel fuel which leaked from the fuel piping of the Salomon Brothers' Standby Generator System on the fifth floor of WTC7, would have compromised Trusses 1 and 2, and would also have initiated the collapse sequence of WTC7, causing failures along the column line of Columns 79, 80, and 81, shown by the sinking of the east penthouse below the roofline.
- 10. Disproportionate collapse of the building interior spread westward due to failure of the transfer trusses and then to the exterior because the cantilevered transfer girders on the north face were supported by one of the transfer trusses. The stacking of critical structural transfer elements created interdependence such that the loss of the transfer truss caused: (1) the cantilevered transfer girders to fail; (2) the perimeter frame to redistribute

load and buckle in the unbraced lower northeast corner of the building, and (3) formation of the "kink" in the north façade visible in the video footage.

- 11. Whether the failure of Columns 79 and/or 80 was initiated by a diesel fuel fire on the fifth floor or an office contents fire between the ninth and thirteenth floors, the horizontal progression and global collapse ensued as a result of one or more of the following omissions: (1) girder to column connections that are weak in tension and did not brace the columns in accordance with the NYCBC requirement that the bracing be able to support 2% of the design vertical load carried by the column; (2) inadequate redundancy in the configuration of the transfer structures; or (3) lack of structural integrity (resistance to disproportionate collapse) in the design and construction of WTC7, including, without limitation, disregard for floor segmentation caused by the trench headers.
- 12. Based on the fire and structural fire engineering analyses that have been performed by others and reviewed by me, and based on my analysis of the global collapse of the structure, it is my opinion that, contrary to established engineering practice, a local failure led to global collapse of the building as a result of the way in which the building was designed and constructed.

I declare under penalty of perjury that the foregoing statements made by me are true. I am aware that if any of the foregoing statements made by me are willfully false, I may be subject to punishment.

GUY NORDENSON

DATED: April 1, 2010

<sup>&</sup>lt;sup>1</sup> Trench headers are hollow ducts located within the depth of a concrete floor slab used for the passage of electrical wiring in an electrified floor system. Had the discontinuities in the concrete floor diaphragms created by the trench headers been addressed by the addition of horizontal bracing, the WTC7's floor system would not have ruptured in the manner it did on September 11, 2001.

Curriculum vita

Name

Guy JP Nordenson

Profession

Structural Engineer

Position

Partner

#### Summary

Guy Nordenson is a structural engineer and professor of architecture and structural engineering at Princeton University. He studied at MIT and the University of California at Berkeley and began his career as a draftsman in the joint studio of R Buckminster Fuller and Isamu Noguchi in Long Island City in 1976. He has practiced structural engineering in San Francisco and New York. In 1987 he established the New York office of Ove Arup & Partners and was its director until 1997, when he began his current practice. In 1993-1994 he was a Loeb Fellow at Harvard University. In 1996 he co-founded the Structural Engineers Association of New York. He was the first recipient of the new American Academy of Arts and Letters Academy Award in Architecture for contributions to architecture by a non-architect in 2003. With Terence Riley he was co-curator of the "Tall Buildings" exhibition held at MoMA QNS in 2004 and his drawings and models for the 2003 WTC Tower 1 design are now in the collection of the MoMA. He is Commissioner and Secretary of the New York City Public Design Commission, the only engineer to serve since the Commission was established in 1898. His project "On the Water - the NY/NJ Upper Bay" won the 2007 AIA College of Fellows Latrobe Research Prize. His Seven Structural Engineers - The Felix Candela Lectures in Structural Engineering was published in 2008 by MoMA. Recently Nordenson was named the William A Bernoudy Architect in Residence at the American Academy in Rome, was a recipient of the AIA's 2009 Institute Honors for Collaborative Achievement Award, and also elected to the American Academy of Arts and Sciences.

Nordenson was the structural engineer for the Museum of Modern Art expansion in New York, the Jubilee Church in Rome, the Simmons Residence Hall at MIT in Massachusetts, the Disneyland Parking Structure in California, the Santa Fe Opera House, and over 100 other projects. Recently completed projects include the New Museum of Contemporary Art in New York, the Nelson-Atkins Museum of Art in Kansas City, the Toledo Museum of Art Glass Pavilion, and the University of Iowa School of Art and Art History. Current projects include the WTC Memorial Museum Slurry Wall bracing structure, 2 pedestrian bridges at Yale University, the Asian Cultural Complex in South Korea, the expansion of the Kimbell Art Museum in Fort Worth and the San Francisco State University Creative Arts Center. Nordenson is also active in earthquake engineering, including code development, technology transfer, long-range planning for FEMA and the USGS, and research. He initiated and led the development of the New York City Seismic Code from 1984 to its enactment into law in 1995.

#### Education

Diploma, Phillips Academy, Andover MA 1973
Baccalauréat, Série C (mathématiques élémentaires) with distinction 1973
BSc, Massachusetts Institute of Technology (Civil Engineering) 1977
MSc, University of California at Berkeley (Structural Engineering & Structural Mechanics) 1978
Loeb Fellow in Environmental Design, Harvard University Graduate School of Design 1993-1994

#### **Affiliations and Qualifications**

Fellow, American Society of Civil Engineers
Founder, and past President, Structural Engineers Association of New York (1996)
Adjunct Curator, Department of Architecture and Design, The Museum of Modern Art, New York (2002-date)
Member, Earthquake Engineering Research Institute (1979), Structural Engineers Association of California (1980),
American Institute of Steel Construction, American Concrete Institute, and Pre-stressed Concrete Institute

Professional Registrations: CA (1980) (Civil & Structural) NY CT PA OH NJ ME HI (Structural) TX NC NM MI TN IA IN

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### **Professional Experience**

1997-date Guy N

Guy Nordenson and Associates LLP, New York NY

#### Current Projects in Design or Construction

National Museum of African American History and Culture, Washington DC (Freelon Adjaye Bond)

Grace Community Church, New Canaan CT (OBRA Architects / Pompei AD)

San Francisco State University Creative Arts Center, San Francisco CA (Michael Maltzan Architects)

Anthology Film Archive Expansion, New York NY (Atelier Ralmund Abraham Architect)

Kimbell Art Museum Expansion, Fort Worth TX (Renzo Piano Building Workshop)

WTC 7 Collapse Investigation, New York NY

Lawrence Convention Center Collapse Investigation, Pittsburgh PA

New York City Police Academy, Bronx NY (Perkins + Will with Robert Silman Associates)

Jeong Dong Building, Seoul SOUTH KOREA (Kyu Sung Woo Architects)

L'Oreal Triangle Site, North Brunswick NJ (Davis Brody Bond)

Ranch House, Eagle View Houses and New York Townhouse Renovation, Red Lion PA and New York NY (Polshek Partnership Architects)

5 Manhattan Pedestrian Bridges, New York NY (Designer and Structural Engineer)

Yale Hillhouse Bridges, New Haven CT (Designer and Structural Engineer)

Asian Cultural Center, Guangju SOUTH KOREA (Kyu Sung Woo Architects)

WTC Memorial Slurry Wall Bracing Structure, New York NY (Davis Brody Bond with Simpson Gumpertz & Heger)

Fehnel Visitors Center Art and Nature Park Walkway Bridge, Indianapolis Museum of Art, Indianapolis IN (Marlon Blackwell Architect, Mary Miss Studio)

Jet Propulsion Laboratories Administration & Education Complex, Pasadena CA (Michael Maitzan Architects)

Linked Hybrid Residential Towers, Beijing CHINA (Steven Holl Architects)

Nanjing Museum of Architecture, Nanjing CHINA (Steven Holl Architects)

#### Completed Projects - Designer and Structural Engineer

Gainsborough Studio Bridge, New York NY 2007

Portsmouth Abbey School Church Restoration, Portsmouth RI 2005 Project

World Trade Center Tower One, New York NY (with SOM) 2003 Project

7 Stems Broadcast Tower, Bayonne NJ (in collaboration with Henry N Cobb/Pei Cobb Freed & Partners) 2002 Conceptual Design

WTC Emergency Building Damage Assessments (SEAoNY, LZA/Thornton-Tomasetti and NYC Dept of Design and Construction). Following 9/11 organized the building damage assessment inspections by SEAoNY teams in September and October 2001.

Disneyland Escalator Canoples, Anaheim CA – Design and Engineering 1999

Fabrications, MoMA New York NY (Installation with TEN Arquitectos) 1998

# Completed Projects - Consulting Structural Engineer

Xochimilco Aquarium and Park, Mexico City MEXICO (TEN Arquitectos) 2008 Project

BAM Two Trees, Brooklyn NY (TEN Argultectos with Robert Silman Associates) 2006-2008 Project

Artreehoose, New Fairfield CT (Della Valle + Bernheimer) 2008

Tropicana Garage Collapse Investigation, Atlantic City NJ (case settled successfully) 2006-2007

New Museum of Contemporary Art, New York NY (SANAA/K Sejima and R Nishizawa) 2007

Nelson-Atkins Museum of Art, Kansas City MO (2000 PA Award - Steven Holl Architects) 2007

Miami Art Museum, Miami FL (Herzog & de Meuron) 2007 Conceptual Design

Toledo Museum of Art Glass Center, Toledo OH (SANAA/K Sejima and R Nishizawa with Sasaki Structural Consultants) 2006

Fresno Metropolitan Museum, Fresno CA (Michael Maltzan Architects) 2005-2006 Project

Visual and Performing Arts Library, Brooklyn NY (TEN Arquitectos) 2004-2006 Project

Ara Pacis Museum Complex, Rome ITALY (Richard Meier & Partners Architects) 2006

Jinhua Pavilion, Jinhua CHINA (Michael Maltzan Architects) 2006

University of Iowa School of Art, Iowa City IA (Steven Holl Architects) 2006

Guggenheim Museum, Guadalajara MEXICO (TEN Arquitectos) 2005 Project

Queens Museum of Art, Queens NY (Eric Owen Moss Architects with Robert Silman Associates) 2005 Project

Goldman Sachs HO, New York NY (Pei Cobb Freed & Partners, with Yolles Partnership) 2004

59 East 59 Theater, New York NY (UrED/Leo Modrein Architect) 2004

Bridges Center, Memphis TN (Building Studio with Coleman Coker Architects) 2004

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Completed Projects - Consulting Structural Engineer (cont)

MoMA Expansion, New York NY (Taniguchi and Associates with Severud Associates) 2004

Bonfire Memorial, College Station TX (Overland Partners with Haynes Whaley engineers) 2004

Newport Office Centers Design Review, Newark NJ (Lefrak Organization) 2004

Jubilee Church, Rome ITALY (Richard Meier Et Partners Architects) 2003

College of Architecture and Landscape Architecture, U of Minn, Minneapolis MN (Steven Holl Architects) 2003

Lehmann Maupin Gallery, New York NY (OMA/Rem Koolhaas) 2003

Nelson-Atkins Museum of Art Parking Structure, Kansas City MO (2000 PA Award - Steven Holl Architects) 2002

MIT Simmons Hall Residence, Cambridge MA (2000 PA and 2003 AIA Honor Awards - Steven Holl Architects) 2002

Anthology Film Archives Heaven and Earth Library, New York NY (Atelier Raimund Abraham Architect) 2002

Oceanic Retreat, Kaual HI (Steven Holl Architects) 2002 Project

Anchor Point Residence, Homer AK (Building Studio/Coleman Coker Architects) 2002 Project

Ferragamo Stores and Cascade/Cantilever Stairs, New York NY, Venice and Bologna ITALY (Michael Gabellini & Associates Architect) 2001

Believue Art Museum, WA (Steven Holl Architects) 2001

Mur River Café and Installation, Graz AUSTRIA (Acconci Studio) 2001

Corning Glass Center, Corning NY (Smith-Miller + Hawkinson Architects - Consultant) 2000

The Umbrella, Culver City CA (Eric Owen Moss Architect) - Consultant 1999

ReyRosa Residence, TX (Building Studio/Coleman Coker Architect) 1999 Project

Disneyland Parking Structure, Anaheim CA (Wolf + Walker Parking Consultants Architects) 1999

Soho Stair, New York NY (2000 AIA Honor Award - Architecture Research Office) 1999

Knut Hamsun Museum, NORWAY (Steven Holl Architects) 1998 Project

Santa Fe Opera House, Santa Fe NM (Polshek & Partners Architects - Consultant) 1998

BDO, Whitney Biennial, New York NY (Glen Seator Artist) 1997

Competitions - Designer and Structural Engineer

River Douglas Bridge Competition, Becconsall UNITED KINGDOM (finalist – 3rd place) 2008
Thu Thiem Bridge and Plaza Competition, Ho Chi Minh City VIETNAM (with Catherine Seavitt Studio and Hargreaves Associates) 2008

Patent Office Building Courtyard Roof Washington DC (with Henry N Cobb/Pei Cobb Freed & Partners) 2004
Sugar House Bridge, Salt Lake City UT (finalist with Catherine Seavitt Studio, Landscape) 2003
Portland Aerial Tramway, Portland OR (finalist with Architecture Research Office) 2003
Stonecutters Bridge, Hong Kong CHINA (finalist with HNTB – Honorable Mention) 2000

Competitions - Consulting Structural Engineer FKI Tower, Seoul KOREA, (Pel Cobb Freed & Partners) 2009 Shenzhen 4 in 1, Shenzhen CHINA, (Steven Holl Architects) 2008 Magok Waterfront and Bridge Competition, Seoul KOREA (Hargreaves Associates) 2008 Perm Museum, Perm RUSSIA (with Acconci Studio) 2008 Guggenheim Museum, Guadalajara MEXICO (TEN Arquitectos - 1st place) 2005 Highline, New York NY (Steven Holl Architects) 2004 City Tower, Chicago IL (Pei Cobb Freed & Partners) 2004 Lombardy Government Center, Milan ITALY (Joint venture with Steven Holl Architects) 2004 Marseilles Museum Competition, Marseilles FRANCE (Steven Holl Architects) 2003 Los Angeles Museum of Natural History, Los Angeles CA (Steven Holl Architects - 1st place) 2002 Los Angeles County Museum of Art, Los Angeles CA (Steven Holl Architects ) 2002 American Craft Museum, New York NY (Allied Works architects - 1st place) 2002 Visual and Performing Arts Library, Brooklyn NY (TEN Arquitectos - 1st place) 2002 School of Architecture, Cornell University, Ithaca NY (Steven Holl Architects - 1st place) 2001 Eyebeam, New York NY (ARO and P Scott Cohen) 2001 Pinault Foundation, Paris FRANCE (Steven Holl Architects) 2001 Burgos Museum of Human Evolution, Burgos SPAIN (Steven Holl Architects) 2000 Nelson-Atkins Museum of Art, Kansas City MO (Steven Holl Architects - 1st place) 1999 Contemporary Art Museum, Rome ITALY (with Steven Holl Architects - 2nd place) 1999 City of Culture, Santiago de Compostela SPAIN (Steven Holl Architects - 2nd Place) 1999 Museum of Modern Art Charrette, New York NY (Steven Holl Architects) 1997 Sapporo Dome Competition, Sapporo JAPAN (Nikken Sekkel and Shimizu - 2nd place) 1997

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1987-1997 Ove Arup & Partners, New York NY
Consultant, Ove Arup & Partners Intl Ltd 1987-1989
Director and Principal, Ove Arup & Partners Consulting Engineers PC 1989-1997
Consultant, Ove Arup & Partners Consulting Engineers PC 1997

Projects -Designer and Structural Engineer

Bridge Over Loiza River, San Juan PR 2002- Competition-winning 240m span single tower cable stayed bridge, 480m long 1996-1997 Project

US Air Canopy, La Guardia Airport, NY (1993 Benedictus Award - in collaboration with Smith-Miller + Hawkinson Architects) 1994 - First US architectural application of advanced composite materials

Completed Projects — Consulting Structural Engineer

Austrian Cultural Institute, New York NY (Raimund Abraham Architect) 2002

Wall Street Esplanade and Ferry Pier, New York NY(Smith-Miller + Hawkinson Architects) 2001

JFK Airport International Arrivals Building, New York NY (SOM Architects - Scheme Only) 2001

Sony HQ, Berlin GERMANY (Murphy/Jahn Architects - Forum Roof scheme only) 2000

Capital Group Companies Offices, San Antonio TX (Pei Cobb Freed & Partners Architects) 1998

Kiasma Museum of Contemporary Art, Helsinki FINLAND (1999 AIA Honor Award - Steven Holl Architects) 1998

Corning Glass Center, Corning NY (Smith-Miller + Hawkinson Architects) 2000
Santa Fe Opera House, Santa Fe NM (Polshek & Partners Architects) 1998
Cranbrook Institute of Science, Bloomfield MI (Steven Holl Architects) 1998
Shorthand House, Houston TX (1997 NY AIA Project Award - Francois deMenil Architect) 1997
Fresco Chapel and Gallery, Houston TX (1994 PA Citation, 1998 NYACE Diamond Award, 1999 AIA Honor Award - Francois deMenil Architect) 1997

Munich Airport Center, Munich GERMANY (Murphy/Jahn Architects) 1997
Neugebauer House, Naples FL (Richard Meier & Partners Architects) 1997
Princeton Children's Library, Princeton NJ (Smith-Miller + Hawkinson Architects) 1997
Mashantucket Pequot Museum, Ledyard CT (Polshek & Partners Architects) 1997
Rachofsky House and Art Gallery, Dallas TX (Richard Meier & Partners Architects) 1997
North Carolina Museum of Art Amphitheater, Raleigh NC (1997 NY AIA Citation - Smith-Miller + Hawkinson Architects) 1997

Sinte Gleska University, Rosebud SD (1996 PA Citation - Roto Architects) 1996
Inventure Place, Akron OH (1999 AlA Honor Award - Polshek & Partners Architects) 1996
Farnsworth Museum, Rockland ME (Toshiko Mori Architect) 1996
Swissair North American Headquarters, Melville NY (Richard Meier & Partners Architects) 1995
Televisa Cafeterias, San Angel and Chapultepec, Mexico City MEXICO (1994 PA Citation and 1998 Mies van der Rohe Latin America Prize - TEN Arquitectos) 1998

Foster Wheeler Manufacturing Plant, Xinhui CHINA 1995
Kuala Lumpur Office Building, Kuala Lumpur MALAYSIA (Tsao & McKown Architects) 1994
660MW Boiler House, Zouxian CHINA – Structural and selsmic design of boiler support structure 1994
American Airlines Terminal Expansion, JFK Airport NY (Murphy/Jahn Architects) 1993 – demolished 2001
Research and development for the seismic design of suspended boiler structures (Foster Wheeler
Energy Corp) 1993

Horseshoe Beach, La Romana DOMINICAN REPUBLIC (Cooper Robertson Architects) 1992 Weatherstone Riding Ring, Sharon CT (Cooper Robertson Architects) 1989 Tokyo International Forum, Tokyo JAPAN (Rafael Viñoly Architect - Schematic Design only) 1989

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1982-1987 Weidlinger Associates, New York NY

Project engineer and from 1985, Associate responsible for new construction, research and

restoration projects:

Completed Projects — Consulting Structural Engineer
Stone Mountain Pedestrian Bridge, Atlanta GA (Jim Fowler)
US Embassies in Nicosia, CYPRUS (KPF Architects) and Damascus SYRIA (Marcel Breuer Associates)
Façade restoration of landmark Dally News Building and 745 Fifth Avenue, New York NY
Principal investigator for NCEER/NSF-funded research "NYC Seismic Design"
Investigation of collapse of HH Humphrey Metrodome, MN

1978-1982 Forell/Elsesser Engineers, San Francisco CA

Project engineer responsible for the design of a number of laboratory and office buildings including two of the first steel eccentric brace frames built in California, Chairman and member of committees of the Structural Engineers Association of California charged with drafting the California seismic code.

1974-1976 Fuller and Sadao & Noguchi Fountains, Long Island City NY

Draftsman and modelmaker for Isamu Noguchi and R Buckminster Fuller. Made models for Fuller portion of the inaugural Cooper Hewitt Museum show "Man Transforms".

Completed Projects

Dodge Fountain, Detroit MI (Noguchi and Sadao)

Samuel Beckett Theater, Oxford UK (Fuller and Sadao and Norman Foster, project)

Teaching Experience

1995-date Princeton University School of Architecture, Princeton NJ

Lecturer 1995-1996, Associate Professor 1997-2000, 2000-2004 (with continuing tenure)

Professor of Structural Engineering and Architecture 2004-date

Faculty Associate, University Center for Human Values, Princeton Environmental Institute and

Department of Civil and Environmental Engineering

Courses

ARC 510 Structural Analysis 1995-date
ARC 518 Construction and Interpretation 1999-date
ARC 408 Infrastructure 2002-date

IWS Undergraduate Studio 1999-date

ARC 501 Introductory Graduate Studio, with Stan Allen, 2007

ARC 505 Art Storage Graduate Studio, with Adam Yarinski and Mahadev Raman, 2006

ARC 504 Stadium Graduate Studio, with Marc Mimram and Catherine Seavitt 2004

CIV 366 Steel and Reinforced Concrete Design 1996-1998

ARC 511 Structural Design 1995-2000

Princeton University Art Museum Director Search Committee 2008
University Campus Planning Steering Committee 2005-2007
School of Engineering and Applied Sciences Strategic Planning 2003
President's Advisory Committee on Architecture 1996-2005
Departmental Representative (in charge of undergraduate program) 1999-2004
Civil Engineering and Architecture Program Committee 1996-date

1995 Massachusetts Institute of Technology, Cambridge MA

Visiting Lecturer, Fall 1995

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1985 – 1995 Columbia University Graduate School of Architecture, Planning and Preservation, New York NY, Adjunct Assistant, then Adjunct Associate Professor

#### Courses

Architectural Consequences of Structural Decisions (with Mario Salvadori) 1985–1987
Structural Design 1987–1995
Mechanisms and Composite Structures (with Chuck Hoberman) 1992
Graduate Studio (with Laurie Hawkinson and Sulan Kolatan) 1989 (with Enrique Norten) 1990
Patterns and Structure 1993, 1995

1983 - 1985 Parsons School of Design, Environmental Design, New York NY, Instructor

#### Sponsored Research

On the Water: The NY/NJ Upper Bay, AIA College of Fellows Latrobe Prize Research Grant and High Meadows Foundation Grant 2007-2008

Princeton Environmental Atlas 2005

New York City Area Consortium for Earthquake Loss Mitigation (NYCEM), Technical Director of 3 year research FEMA project to develop GIS based model for earthquake loss estimation in the New York City area 1998-2002

#### Awards

Fellow, American Academy of Arts and Sciences, Cambridge MA 2009

AlA Institute Honors for Collaborative Achievement, San Francisco CA 2009

William A Bernoudy Architect in Residence, American Academy in Rome, ITALY December 2008 – February 2009

Premio Mario Pani Award, School of Architecture University of Anahuac, Mexico City MEXICO 2008

AlA College of Fellows Awards 2007 Latrobe Prize for "On the Water, A Model for the Future: A Study of New York and New Jersey Upper Bay" research project (http://www.aia.org/release\_031207\_Latrobe)

Academy Award in Architecture, American Academy of Arts and Letters, New York NY 2003

First awarding of a new AAAL award for contribution to architecture by a non-architect IDCA Fellow, International Design Conference, Aspen CO 1995

National Science Foundation Travel Grant to Tokyo JAPAN1988

Northern California AISC steel design award 1982

### Lectures

Bridges, Studio 360, New York NY 2007
 Visiting Lecturer/Critic: Yale, Pratt Institute, U of MN, NJIT, UCLA, Princeton, VA Tech, MIT, Iowa State, Columbia, GA Tech, U of FL Gainesville, Cornell, IIT, Am Bar Assoc, U of VA, Sci Arc, Arch League of NY, Wash U, Temple U, Syracuse U, ASCA (2005 Keynote), Harvard, AIA Fort Lauderdale
 Walker Art Center Summer Lecture Series, Minneapolis MN 1991
 Arthur H Schein Memorial Lecture (with Smith-Miller + Hawkinson, MIT Department of Architecture) 1991
 Witness, House of Representatives Subcommittee on Science, Space and Technology "Lessons Learned from the Northridge EQ", Washington DC 1994

## **Exhibitions**

Slurry Wall, photograph, AIA Center for Architecture, New York NY 2006
On the Water – NY NJ, video, with Aaron Forrest, 10thVenice Architecture Blennale, Venice ITALY 2006
Camber, Sci-Arc, Los Angeles CA 2004
Selected Works, Academy of Arts and Letters, New York NY 2003
Sketchbooks, Estes Gallery, Memphis TN 2003
NEXT – B Mostra Internationale de Architettura-la Blennale de Venezia, Venice ITALY 2002
Art on the Beach, with Jody Culkin and Uwe Mengel, Creative Time, New York NY 1988

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#### **Publications**

Books

On the Water | Palisade Bay, with C Seavitt and A Yarinsky, www.lulu.com, New York NY 2009

Seven Structural Engineers - The Felix Candela Lectures in Structural Engineering, editor, MoMA Publications, New York NY 2008

New York Consortium for Earthquake Hozard Mitigations, Summary Report with M Tantala et al, MCEER Publication, Buffalo NY 2003

Tall Buildings, with Terence Riley, MoMA Publications, New York NY 2003

WTC Emergency – Damage Assessment of Buildings Structural Engineers Association of NY Inspection of September and October 2001 Volume A Summary Report, and B-F on DVD, SEAONY, New York NY 2003

## Research Reports

'Earthquake Loss Estimation Study for the New York City Metropolitan Region', with MW Tantala, G Deodatis and KH Jacob, Journal of Soil Dynamics and Earthquake Engineering, October 2007

'Earthquake Loss Estimation Study for the New York City Area', Final Report, Princeton University, School of Architecture and Dept of Civil and Environmental Engineering, funded by FEMA, Jan 2003

'Earthquake Loss Estimation Study for the New York City Area', Second Year Technical Report, Princeton University, Dept of Civil and Environmental Engineering, submitted to MCEER and funded by FEMA, Jan 2000

'Earthquake Loss Estimation for the New York City Area' with G Deodatis, KH Jacob and MW Tantala Technical Report Prepared for MCEER, Princeton University, Princeton NJ 1999

Seismic Hazard Evaluation for New York City Report of the NYACE Ad-hoc Seismology Comm New York Oct 1986

#### Articles

'Infrathin' in Engineered Transparency-The Technical, Visual, and Spatial Effects of Glass, editor Michael Bell and Jeannie Kim, Princeton Architectural Press, New York NY 2009

'On The Water: The New York-New Jersey Upper Bay', with S Cassell, M Koch, C Seavitt, J Smith, MW Tantala and A Yarinsky in Places, 02 November 2008

'Duelling Partners' in The Architect's Newspaper, 09 July 2008

'Glass Pavilion, Toledo Museum of Art', with Brett Schneider in Structural Engineering International, February 2008 'Apocryphal' in Domus, December 2007

Freedom From Fear' in The New York Times, 16 February 2007

'Building Bridges', with Noah Klersfeld and Jiro Takagi in Civil Engineering, February 2007

'Concrete Theater' in Liquid Stone: New Architecture in Concrete, editor Jean-Louis Cohen and G Martin Moeller, Princeton Architectural Press, New York NY 2006

"With Great Joy and Expectations", for Noguchi- Fuller exhibit catalog, Noguchi Museum, Long Island City NY 2006 "Tall Buildings" Biennale de Venezia Catalog, Venice ITALY 2002

'City Square: Structural Engineering, Democracy and Architecture' Grey Room 7, New York NY 2002

The Daily Practice of Collaboration' introduction to Architecture Research Office, Princeton Architectural Press, New York NY 2002

'Earthquake Loss Estimation for the New York City Area' with G Deodatis, KH Jacob and MW Tantala, 7th National Conference on EQ Engineering (7NCEE), EQ Engineering Research Institute (EERI), Boston MA July 2002

'Collaboration' Perspecto 31: Reading Structures, Yale Architecture Journal, New Haven CT 2000

'4 Experimental Projects' Dialogue, Talpei TAIWAN 2000

'Selsmic Design Procedures for Regions of Moderate Seismicity' with GR Bell, Earthquake Spectra, Feb 2000 vol 16 no 1, Oakland CA

'Seismic Design Requirements for Regions of Moderate Seismicity' with GR Bell, Proc 12th World Conference in EQ Engineering, Auckland NEW ZEALAND 2000

'Earthquake Loss Estimation for the New York City Area' with G Deodatis, KH Jacob and MW Tantala Proc 12th World Conference in EQ Engineering, Auckland NEW ZEALAND 2000

'The Lineage of Structure and the Kimbell Art Museum' Lotus 98, Milan ITALY 1998

'Notes on Bucky / Patterns and Structure' in ANY 17, New York 1997

'Notes on Light and Structure' in Light in Architecture, Architecture Review, London UK Apr 1997

'Critical Mass' in Das Grosse-On Bigness, Daldalos 61, Berlin GERMANY Sept 1996

'Built Value and Earthquake Risk' Proc NCEER Conf Economic Consequences of Earthquakes: Preparing for the Unexpected, New York NY Sept 1995

Time and Section Study' on Santiago Calatrava in Columbia University Newsline, New York NY 1993

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#### Articles (cont)

The Spirit of Measure' Introduction to Harry Wolf, Editorial Gustavo Gili SA, Barcelona SPAIN 1993
'Seismic Codes' in Monograph 2 on the Mitigation of Damage to the Built Environment, National Earthquake
Conference, Memphis TN 1993

'An Inventive Nature' on Chuck Hoberman in Sites, New York NY 1991

'Earthquake Hazard Reduction in Urban Areas of Moderate Seismicity' 3rd US-Japan Workshop on Urban Earthquake Hazard Reduction, Honolulu HI November 1991

'Adapting Seismic Codes for Zones of Moderate Seismicity: the New York City Experience' New Jersey Section, ASCE Oct 1990

'Acceptable Damage in Low and Moderate Seismic Zones' with LD Reaveley, ATC 15-3 4th US-Japan Workshop on the Improvement of Building Structural Design Practices, Kailua-Kona HI Aug 1990

'Seismic Design of Suspended Boiler Structures' with PJ Donelan and M Garkawe Proc 4th US National Conf on Earthquake Engineering, Palm Springs CA May 1990

'Evaluation of Earthquake Resistance of Existing Building Practice in New York City' Proc 9th World Conf on Earthquake Eng, Tokyo JAPAN 1988

"Wind versus Seismic Design' Earthquake Hazards and the Design of Building Facilities in the Eastern United States New York Academy of Sciences, New York NY Feb 1988

'Some Limitations of Current Seismic Codes for Eastern US Earthquake Resistant Design' Proc Symp on Seismic Hazards, Ground Motions, Soll Liquefaction and Engineering Practice in Eastern N America, Sterling Forest NY 20-22 Oct 1987

'Seismicity and Seismic Hazard in the New York City Area' with CT Statton Proc 3rd US Not Conf. on Earthquake Eng. Charleston SC 1986

'Review of Current and Proposed US Seismic Codes for Steel Structures' Proc ECCS-IABSE Symp Steel in Building, LUXEMBOUG 1985

'Notes on the Seismic Design of Steel Concentrically Braced Frames' Proc 8th World Conf on Earthquake Eng. San Francisco CA 1984

'BSSC Trial Design Program-Bulldings NY-5, NY-20A and NY-32' Weidlinger Associates Report to the Nat Inst of Bldg Sci/Bldg Seismic Safety Council No 182-016 'ATC-3-06 Trial Design Program', NY 1984

'Aseismic Reinforcement of Existing Buildings' with NF Forell Jinl of the Struct Div Proc ASCE Vol 106 No ST9,1980 Rune, the MIT Arts & Letters Magazine, Cambridge 1977-present, founding editor

## **Books and Articles about**

'Action and Practice' in Perspecta 40: Monster, Yale Architecture Journal, New Haven CT 2008
David W Dunlap, 'For 9/11 Wall, a Little Support and a Permanent Place' in The New York Times, 28 April 2008
Joann Gonchar, 'Behind SANAA's Illusion of Weightlessness' in Architectural Record, March 2008
Nina Rappaport, 'Guy Nordenson and Associates' in Support and Resist - Structural Engineers and Design
Innovation, Monacelli Press, New York NY 2007

Joann Gonchar, 'Glass: Transparent, Translucent, and Ironic' in Architectural Record, 15 October 2007 'New Museum of Contemporary Art: Steel Balancing Act' in Metals in Construction, Fall 2007

Nina Rappaport, 'The Engineer's Moment' in Architectural Record, August 2007

Jane F Kolleeny, 'Guy Nordenson Sketches to Think' in Architectural Record, August 2007

John E Czarnecki, 'AlA Awards Latrobe Prize to Flood Research' in *Architectural Record* (Online), 3 May 2007 Karen Trimbath, 'Engineering Collaboration Provides Structure for Glass Pavilion' in *Civil Engineering*, November 2006

Frederic Edelmann, 'Querelles autour du projet de Ground Zero' In *Le Monde*, 9 February 2005 Martin Filler, 'Filling the Hole' in *The New York Review of Books*, 24 February 2005, Vol 52 No 3 Suzanne Stephens, 'Museum of Modern Art, New York' in *Architectural Record*, January 2005 Glenn Collins, 'Public Lives: Behind a Graceful Spire, Science, Art and Passion' in *New York Times*, 29 December

Herbert Muschamp, 'A Skyscraper Has a Chance To Be Nobler' in New York Times, 20 December 2003
Lynne Duke, 'Revised WTC Freedom Tower Design Unveiled' in Washington Post, 19 December 2003
Adam Gopnick, 'Higher and Higher – What tall buildings do' in New Yorker, 15 December 2003, review of 'Tall Buildings'

David W Dunlap, 'BLOCKS: The New Look at Ground Zero May Be the Oldest' in New York Times, 11 December 2003 David W Dunlap, 'Plans Reveal World's Tallest Tower, But Only 70 Stories Will Be Inhabited' in New York Times, 10 December 2003

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Books and Articles about (cont)

Randy Kennedy, Threats and Responses: the Brooklyn Bridge: A Conspicuous Terror Target Is Called Hard to Topple' in New York Times, 20 June 2003

Julie lovine, 'Hard Hat Special: Slide Rule Set, Nameless No More' in New York Times, 30 January 2003
Herbert Muschamp, 'A See-Through Library of Shifting Shapes and Colors' in New York Times, 19 January 2003
James Glanz, 'Quietly Professionals Review High Rise Safety After 9/11' in New York Times, 23 October 2002
James Glanz, 'Comparing 2 Sets of Twin Towers: Malaysian Buildings offered as a Model' in New York Times,
23 October 2002

David W Dunlap, 'Designing Buildings to Resist Earthquakes' in New York Times, 30 June 2002.

Herbert Muschamp, 'Thinking Big, a Plan for Ground Zero and Beyond' and 'Don't Rebuild, Reimagine' in NYT Magazine, 8 September 2002, www.nytimes.com/library/magazine/home/20020908mag-index.html

Sara Hart, 'Dynamic Concrete in the 21st Century' in Architectural Record, October 2001

Virginia Fairweather, 'Santa Fe Sensation' in Civil Engineering, May 1998

Sara Hart, 'The Engineer's Hand' in Architecture, November 1998

# Committees, Juries and Directorships

New York City Green Codes Task Force, Climate Adaptation Technical Committee, 2008-date
Architect's Committee for the Far West Side, Regional Plan Association, 2007-date
Harvard University Presidential Ad Hoc Committee 2008-date
Commissioner and Secretary, New York City Public Design Commission 2006-date
Urban Age Advisory Board - a joint initiative of the London School of Economics and the Alfred Herrhausen
Foundation for International Dialogue 2005-date
Building Committee, 101 Spring Street, Judd Foundation NY 2004-date
Earthquake Engineering Research Institute (EERI) Spectra, Editorial Board 2002-date
EERI Design Series, Editorial Board 1992-date

#### Past

US GSA National Register of Peer Professionals 2002-2004
NY City Dept of Buildings WTC Building Code Task Force 2002
ASCE Electronic Computation Committee and Subcommittee on Structural Control 1992-date
National Earthquake Hazards Reduction Program (BSSC/FEMA) Technical Committee TS-1 1997-2000
ATC-35/USGS Ground Motion Initiative 1997-2001
GSA Design Awards Jury 2000
Due Ponti Pedonali Jury, Rome ITALY 2000
EERI Technical Advisory Committee, FEMA Strategic Plan 2005, 1997-1999
Architectural League of New York, Director (1989-1997) and Vice President for Engineering (1993-1997)
ASCE Electronic Computation Committee and Subcommittee on Structural Control 1992-1997NY State Earthquake Advisory Committee 1990-1997
Earthquake Engineering and Risk Workshop USGS Earthquake Hazards Program 5-year plan Jan 1997

Earthquake Engineering and Risk Workshop USGS Earthquake Hazards Program 5-year plan Jan 1997 USGS/FEMA/BSSC 'Project '97' BSSC Seismic Design Procedure Group 1995-1997

NY City Dept of Buildings Code Committee, Chairman 1989-1996

ATC-34 'Critical Review of Current Approaches to Earthquake Resistant Design' Project Engineering Panel 1994 ATC-33 'Guidelines for the Seismic Rehabilitation of Buildings' Ground Motion/Loads and Force

Technical Team 1994-1996

ATC-35 'Workshop on National Ground Motion Mapping' Risk Representation Working Group 1995 USA Presidential Design Awards, panelist and chair, engineering panel 1995

American Academy in Rome, Year of Architecture Committee 1994-95

AIA Awards Jury, St Louis Chapter 1992

International Journal of the Structural Design of Tall Buildings, Editorial Board 1992
ASCE Task Comm on Evaluation and Correction of Seismic Deficiencies in Existing Buildings

5th US-Japan Workshop on the Improvement of Building Structural Design Practices Steering 1992

3rd US Japan Workshop on Urban Earthquake Hazard Reduction Steering Committee 1991

The Real Estate Board of NY Design Committee 1990

NY State Council on the Arts Capital Funding Initiative Panelist 1989-1991

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#### Post (cont)

4th US National Conference on Earthquake Engineering Technical Program Committee 1990 ATC-24 'Development of Standardized Procedures for Seismic Testing of Components of Steel Structures' Project Engineering Panel 1989, 1991

Structures' Project Engineering Panel 1989-1991
ATC-21 'Rapid Visual Screening of Buildings for Potential Seismic Hazards: a Handbook' and ATC-22 'Development of a Handbook on Seismic Evaluation of Existing Buildings' Technical Advisory Committees 1988

ATC-15-2 '3rd US-Japan Workshop on Improvement of Building Structural Design and Construction Practices' US Delegation 1988

NY Association of Consulting Engineers Ad-hoc Seismology Committee Chairman 1984–87 SEAONC Research Committee and SEAOC Technical Activities Committee Chairman 1981–82 SEAOC Seismology Committee 1980-date

200905

JA-3899

Due to its size, Exhibit B has not been e-filed. If you wish to receive a copy, please email Marilyn Francisquini at <a href="mailto:mfrancisquini@greenbaumlaw.com">mfrancisquini@greenbaumlaw.com</a>. A hardcopy of Exhibit B has been filed with the Court, and has been served upon the following counsel:

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REPORT AND SUMMARY OF FINDINGS: GLOBAL COLLAPSE ANALYSIS

# WORLD TRADE CENTER 7 COLLAPSE INVESTIGATION New York NY

Prepared for

Gennet, Kallmann, Antin & Robinson PC and Greenbaum, Rowe, Smith & Davis LLP

12 February 2010

Ву

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B.0	APPENDIX B – Floor Collapse Analysis Report
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E.0	APPENDIX E - Expert CV, Compensation and Prior Expert Work
F.0	APPENDIX F – List of Sources Relied Upon in Formulation of Opinion

## 1.0 EXECUTIVE SUMMARY

On 11 September 2001, debris from the collapsing World Trade Center Tower 1 impacted the World Trade Center 7 office building (WTC7) causing only exterior structural damage to the south face and southwest corner of the building and igniting fires in the building. The fires traveled through several floors of the building over the course of the day. At approximately 5:21pm, the East Penthouse of the building fell, indicating failure of the interior structure on the east side of the building. Approximately five seconds later, the entire building collapsed. The results of structural fire studies documented in Dr Colin Bailey's report indicate that the failure of a single floor girder on a lower floor of the building due to the effects of fire initiated the building collapse.

A well-designed building should have sufficient structural integrity to withstand a local failure such as the loss of a single girder with only local consequences. However, as a result of deficiencies in both its overall design and its details, the WTC7 structure lacked redundancy and robustness and therefore did not have sufficient resistance to disproportionate collapse. Its design lacked a fundamental consideration for structural integrity and load path redundancy.

The primary vulnerability of the building was the fact that the steel floor framing connections to 18 of the 24 interior columns (and 46% of all the floor-to-interior column joints) in the building failed to comply with the lateral bracing requirement for columns in Section C26-1001.2 of the Building Code of the City of New York. The prevalence of double-angle "knife" connections welded to interior columns combined with the frequent use of three-sided column bracing resulted in many locations where interior columns were not adequately laterally-supported. It is evident that the design team did not consider the lateral bracing code requirement in the design of the building because no direction was provided in the contract documents to the contractor's fabricator to design the connections for axial loads. Furthermore, simple hand calculations, had they been performed, would have demonstrated that it is impossible to design a welded double-angle connection for the tensile loads required to brace the heavily-loaded columns in the WTC7 building. As a result of the extensive use of these "knife" connections, the building was in a tenuous state prior to the initiating local collapse event because many of the interior columns were already vulnerable to buckling.

In addition to the pervasive lateral bracing code violations, other characteristics increased the susceptibility of the building to disproportionate collapse. These include the presence of multiple interconnected transfer structures, the use of trench headers in the floor slabs and the large tributary floor areas of interior columns. The use of numerous, and in some cases interdependent, transfer structures with no secondary load path or redundancy, created an interdependency of the structure that made it virtually impossible for a local collapse to remain local. The trench header ducts distributed

throughout the building disrupted the continuity and integrity of the concrete floor diaphragms. The long-span floor framing and large column tributary areas amplified the potential for damage from a single local failure. In this respect, the structure was designed with little consideration of the established standards for structural integrity and the prevention of disproportionate collapse.

Although the precise details of the collapse cannot be exactly simulated by a computer analysis, the probable¹ stages of the collapse can be identified using information gained from computations evaluating the effects of the known vulnerabilities in the structure combined with an analysis of the visual evidence of the collapse. In contrast to other "black box" analysis programs that could be used to study the collapse, the combination of hand calculations and straightforward computer models presented in this report provides a transparent and easily verifiable account of the collapse.

The probable global collapse sequence is summarized in six stages (Figure 1.1):

- 1 Following the unseating of Girder 44–79 at Column 79 due to fire effects, Stage 1 consists of the progressive collapse of the floor areas in the northeast corner of the building to the ground.
- 2 In Stage 2, Column 79 buckles as a result of the loss of adjacent floor structure due to its inadequate lateral bracing.
- 3 In Stage 3, floor loss from the buckling of Column 79 triggers the buckling of Column 80 which is also inadequately laterally braced.
- 4 In Stage 4, collapsing floor slabs trigger the failure of Transfer Trusses 1 and 2 and the subsequent failure of two deep transfer girders, leading to extensive additional interior floor collapse and the buckling of the northeast corner of the perimeter frame.
- 5 In Stage 5, the buckling of the perimeter frame spreads to the south and west. At the same time, the falling interior floor slabs cause the remaining intact floor diaphragms to rupture along their trench headers resulting in lateral displacements that cause twelve interior columns to buckle simultaneously.
- 6 In the final stage of collapse, falling floors fail five transfer girders and the buckling of the perimeter frame continues to the west overloading the remaining perimeter supports and resulting in the total collapse of the building.

<sup>&</sup>lt;sup>1</sup> When used in this report as part of an opinion, the word "probable" means "to a reasonable degree of scientific probability"

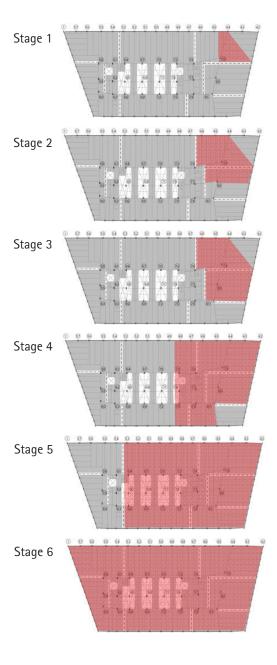


Figure 1.1 Stages of global collapse

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It is apparent from this sequence of events that the lateral bracing code violations at Columns 79 and 80 as well as the presence of multiple, interdependent non-redundant transfer structures were directly responsible for the progression from a local girder failure to a global collapse on 11 September 2001. Had Columns 79 and 80, carrying unusually large tributary load due to long floor bays, been designed with the code-required 2% lateral bracing, these columns would have been able to withstand the adjacent northeastern floor failure, and the collapse on 11 September 2001 would have been arrested after the lower floor failures in the northeast corner of the building (Stage 1). Furthermore, had the transfer trusses been designed with additional redundancy, it is probable that the collapse could have been arrested at Stage 3.

Although the global collapse occurred in the specific sequence described above, because the structure's deficiencies were so pervasive, in my opinion, disproportionate collapse, or total collapse, would likely have resulted from the failure of a single girder in numerous other locations in the building. In this respect, the WTC7 structure was akin to a house of cards, and its global collapse on 11 September 2001 was not unique to the events of the day.

## 2.0 INTRODUCTION

This report summarizes the engineering analysis and findings of Guy Nordenson PE SE regarding the probable cause of the global collapse of the World Trade Center 7 office building (WTC7) on 11 September 2001 and the inherent vulnerabilities of the structure that made it susceptible to disproportionate collapse.

# 2.1 Description of Structure

#### 2.1.1 General

WTC7 was a 47-story steel office building designed by the architect Emery Roth and Sons PC and the structural engineer Irwin G Cantor PC ("Cantor"). The applicable building code at the time of the design was the 1968 New York City Building Code. The building was trapezoidal in plan as shown in Figure 2.1. The north face of the building was approximately 323 feet in length; the south face of the building was approximately 244 feet in length; and the sides of the building were approximately 148 feet wide. The approximate height of the building excluding the roof penthouses was 612 feet. The building, completed in 1987, was constructed by Tishman Construction Corporation over an existing 3-story Con Edison substation built in 1967.

#### 2.1.2 Gravity System

The gravity system consisted of steel columns and floor framing with concrete slab on metal deck. The interior columns were widely spaced in order to maintain an open floor plan and to limit column interference with both the existing substation below and the truck ramp on the east side of the building at grade. The use of widely-spaced interior columns resulted in uncommonly long spans for the floor framing, up to 53 feet. Columns 79 and 81 on the east side of the building supported especially large tributary areas. These two columns alone supported approximately 8% of the floor area of the building.

Steel girders and floor beams ranged from W12's to W36's with the exception of several built-up plate girders. The floor beams (ie secondary framing members which spanned between girders) were designed to act compositely with the concrete floor slab through shear studs. The girders (ie primary members which spanned between columns) were originally designed to be non-composite, although there is evidence that shear studs were added to a number of these members to increase their load carrying capacity during construction (refer to Section 3.5.1 and Appendix D).

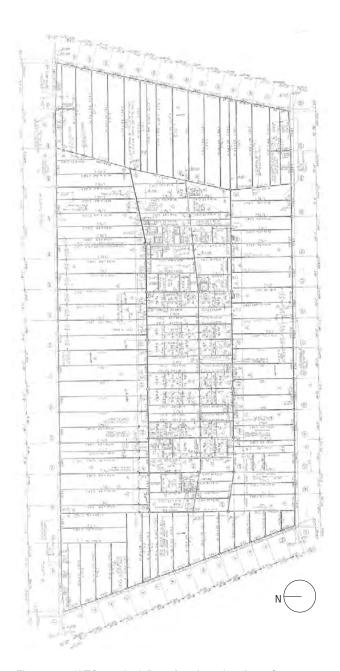


Figure 2.1 WTC7 typical floor framing plan from Cantor structural drawings (TISHMAN014724)

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The layout of the floor framing was generally consistent above Floor 7 with floor beams spanning from perimeter columns to interior columns and the inner core framed with north-south oriented floor beams. The use of both north-south and east-west oriented floor beams created corner conditions where floor beams framed into girders from only one side, rather than two sides (Figure 2.2). The trapezoidal shape of the building also resulted in a number of skewed girder-to-column connections. These aspects of the design were material to the local collapse detailed in the report by Dr Colin Bailey.

The steel columns consisted of W14 wide-flange sections which were in some cases built up with additional web, flange and side plates on the interior and exterior columns of the lower 22 floors. The columns either extended to the ground and were supported on new caisson foundations or were supported on the existing columns and caisson foundations of the Con Edison substation.

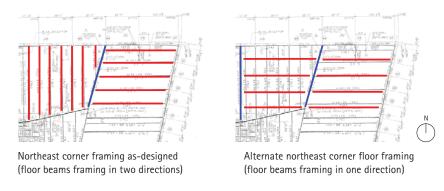


Figure 2.2 Configuration of typical floor framing at northeast corner

## 2.1.3 Connection Details

According to the steel shop drawings, the primary connection details between the girders and columns fall into several classifications: double-angle "knife" type connections (bolted to girder and welded to column), double-angle "header" type connections (welded to girder and bolted to column), and seated connections. Figure 2.3 provides an illustration of each type of connection.

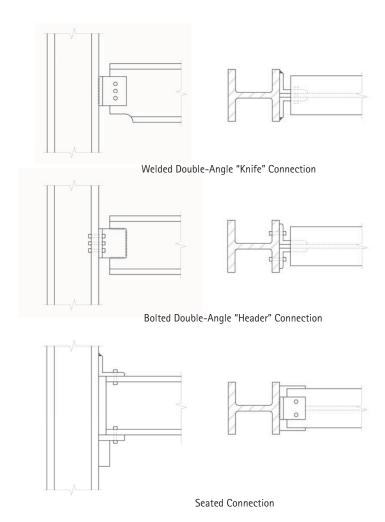


Figure 2.3 Primary girder-to-column connection types (shown in elevation and plan-section)

# 2.1.4 Transfer Structures

In order to resolve the column layout of the upper levels of the building with the layout of the existing columns in the Con Edison substation, the designers used a number of transfer structures on the lower floor levels of the building. These included three transfer trusses between Floors 5 and 7; eight deep, built-up transfer girders between Floors 6 and 7; and several additional transfer girders on Floors 5 and 7 (Figure 2.4).

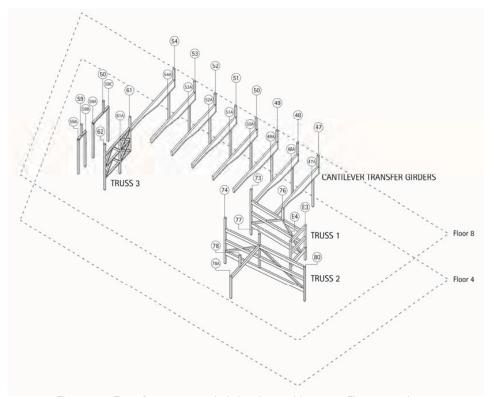


Figure 2.4 Transfer trusses and girders located between Floors 5 and 7

# 2.1.5 Lateral System

The lateral system consisted of perimeter moment frames on all four facades of the building above Floor 7 as well as on the lower seven stories of the north and south sides of the building. The wind girders forming the perimeter moment frames were W36's which were bolted to W14 perimeter columns. Two perimeter belt trusses at Floors 5 to 7 and 22 to 24 provided additional lateral stiffness to the system. Braced frames were used in place of moment frames on the lower seven stories of the building on the narrower east and west sides (Figure 2.5). Additionally, the inner core of the building had both concentric and eccentric braced frames over the lowest seven floors. A thickened reinforced concrete slab floor diaphragm and a horizontal truss at Floor 5 transferred lateral loads from the perimeter to the core. Trench header ducts for electrical wiring disrupted the continuity of the concrete floor diaphragms on the majority of the floor levels.

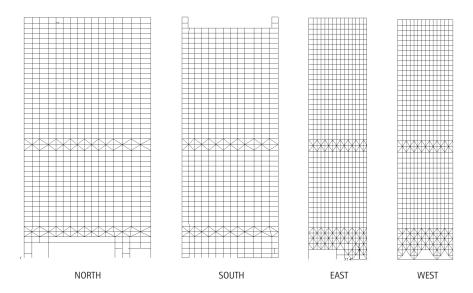


Figure 2.5 Structural elevations of WTC7

### 2.1.6 Tenant Fit-Out Work

Following the completion of construction in 1987, additional tenant fit-out work by Skidmore Owings and Merrill (SOM) with structural engineering consulting from Irwin G Cantor PC took place in 1989. This work mainly included the reinforcement of floor beams and girders with flange plates to carry extra live load and the addition of numerous large web penetrations in floor beams and girders to allow for the passage of mechanical and electrical duct work. Additionally on several floors, floor slabs were removed to create double-height spaces.

## 2.2 Description of Collapse on 11 September 2001

On 11 September 2001, debris from the collapsing World Trade Center Tower 1 impacted WTC7, which was in the process of being evacuated, and caused impact damage to the south face and southwest perimeter corner of the building. Fires were ignited by the debris. The fires germane to this report then subsequently traveled through the building on multiple floors between Floors 5 and 13 over the course of several hours (note – fires above Floor 13 did not contribute to the collapse). Videos obtained from that day show that at approximately 5:21pm the East Penthouse of the building fell, indicating collapse of the interior structure on the east side of the building. Approximately 5 seconds later

the videos show that the interior collapse progressed westward, almost simultaneously with the collapse of the perimeter frame. Videos taken from the north side of the building show that during the collapse, the top of the building remained approximately horizontal. The only notable change in the building's geometry was the creation of a horizontal 'kink' on its north façade (see Section 5.1).

# 2.3 Description of Global Collapse Analysis Approach

The approach used in the global collapse analysis was to identify the effects of innate vulnerabilities of the structure to disproportionate collapse and to use them in combination with the visual indicators of collapse from video footage to determine the primary sequence of events that led to the global collapse of the building on 11 September 2001.

The global collapse studies employed a static structural analysis computer model of the entire building which was deconstructed in stages from the initiating event to the final stages of global collapse. The global model was used to track the structure's loads and deformations at each individual stage. This data was then used as input for independent analyses using more detailed sub-models. The results of the analysis of each detailed sub-model or calculation were then used to inform the next stage of deconstruction within the global model (Figure 2.6).

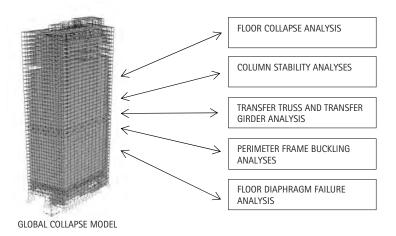


Figure 2.6 Global collapse model and sub-model interface

Both hand calculations and straightforward computer models were used in the studies, all of which were based on the first principles of physics. These analyses provide a transparent and easily verifiable account of the vulnerability of the structure and of the collapse that occurred on 11 September 2001.

## 2.3.1 Global Model Description

The global structural model was built and analyzed in SAP2000 Advanced Version 12.0.2 (Ref 17), a structural finite element analysis program developed by Computers and Structures Inc of Berkeley CA with a Staged Construction module which allows for specialized nonlinear static analysis.

The global model included only the structure that was built above or adjacent to the existing Con Edison substation from 1985 onwards. Although the Con Edison substation was not modeled, relevant information regarding its foundations and other structural details was reviewed and deemed to have no influence on the collapse study.

Frame elements were used to represent beams, columns and braces, and shell elements were used to represent floor slabs. The bending stiffness properties of the shell elements were modified to achieve one-way load distribution consistent with the ribbed metal decking. The vertical offset between the floor framing and the floor slabs was modeled, and where the framing was composite with the floor slab, the stiffness due to the offset was accounted for. The level of meshing for both the frame elements and shell elements was limited to the extent that it would ensure accurate results but not unnecessarily increase analysis run-time (Figure 2.7).

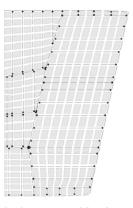


Figure 2.7 Floor slab element meshing in partial view of global model

The global collapse model of the full building was analyzed elastically. Because the global model was elastic, nonlinearities and their effects were taken into account in the more detailed sub-models, which informed the progression of collapse in the global model. Additional key assumptions of the global model, including an explanation of the document review process that formed the basis for the construction of the global model, are described in Section 3.0.

#### 2.3.2 Sub-Model Description

A number of detailed sub-models were built and analyzed in SAP2000 Advanced Version 12.0.2 including models to study the stability of individual core columns and to analyze the vulnerability of the diaphragm at trench headers. The load input for the sub-models was taken from the global model. The sub-models were smaller than the global model and therefore better equipped to handle a greater level of detail including finer meshing and the use of "Link" elements with nonlinear force-displacement relationships. Sections 4.0 and 5.0 provide more detail on these studies.

#### 2.3.3 Conservative Assumptions

Where factual evidence was not available, assumptions that were beneficial to the capacity and performance of the structure were used as the basis for both the global analysis model and the detailed sub-models. These assumptions are explained in greater detail in Section 3.0 but are summarized as follows:

- Loading was a lower-bound estimate that included the tenant fit-out changes to the structure that would have decreased loads (ie floor slab removal). Other changes to the structure which tended to increase loads (ie the plating of girders) were considered and concluded to have no effect on the identified collapse mechanism except to make it more severe and even more expected
- Capacities of structural members were increased from design strengths to either expected strengths or average actual strengths from material test reports
- Except at the northeast corner of the building where remaining slab area is explained by the floor failure mechanism, no hanging slabs were assumed to load the perimeter frame following floor failure

## 3.0 ANALYSIS BASIS

This section outlines the key assumptions that formed the basis of the global collapse analyses documented in this report. Additional assumptions that are specific to individual analyses are described in the corresponding sections of the report.

#### 3.1 Document Review and Use

Guy Nordenson and Associates (GNA) performed a comprehensive review of the material documents related to the design and construction of WTC7. A detailed list of these documents with corresponding Bates numbers is provided in Appendix F. They include but are not limited to the following:

- World Trade Center 7 Structural Drawings by Irwin G Cantor PC and revisions
- World Trade Center 7 Architectural Drawings by Emery Roth and Sons PC Architects
- World Trade Center 7 Electrical, Mechanical and Plumbing Drawings
- Salomon Brothers Tenant Fit-Out and Alteration Architectural and Structural drawings by Skidmore Owings and Merrill and Irwin G Cantor PC
- Structural Steel Erection, Shop and Fieldwork Drawings by Frankel Steel Limited
- Structural Steel Shop Drawings by Steel Structures Corporation
- Fieldwork drawings by Frankel Steel Limited
- Metal Deck Shop Drawings by Nicholas J Bouras
- Floor Trench Shop Drawings by Mac Fab
- Port Authority of New York and New Jersey Specifications for Structural Steel,
   Concrete Slabs and Metal Decking
- Testwell Craig Laboratories testing and inspection reports for concrete, welding, spray-on fire protection, and structural steel erection
- Mill Test Reports from US Steel Co, Stelco, Bethlehem Steel, Algoma, and British Steel Co
- Miscellaneous correspondence, sketches, and calculations issued by the Office of Irwin G Cantor, Frankel Steel, Tishman Construction, Silverstein Properties and other parties
- Contractor change orders related to structural steel, shear studs, metal deck, concreting, foundations and fire proofing
- Deposition transcripts of various parties
- Video footage of WTC7 collapse

GNA catalogued almost 4,000 of the reviewed WTC7 design drawings, shop and erection drawings and change orders in a document database using Microsoft Access. The document data were inputted such that the database is searchable by categories including author, recipient, date, trade, steel member type, and floor level. The purpose of the database was to verify that the global model and associated sub-models were

built and analyzed using the most relevant and recent information regarding the WTC7 structure. Additional documents that were received after the creation of the database were reviewed and confirmed with reasonable certainty to not alter the assumptions and conclusions of the global collapse analyses.

#### 3.2 Loading Assumptions

## 3.2.1 Floor Dead and Superimposed Dead Loads

The dead and superimposed dead floor loads used in the global collapse analyses were based on the Cantor design documents, calculations and construction correspondence using a conservative, lower-bound approach. Confirmed changes made to the structure during the original construction, such as the plating of certain floor beams, were included in the loading of the model. However, unconfirmed changes made to the structure during the original construction, such as the addition of a 10 psf dead load as indicated in construction correspondence, were not accounted for in the loading of the model. Similarly, changes made to the structure during the subsequent tenant alterations that increased loading to the structure were not included in the model, while changes to the structure that decreased loading, such as floor slab removals, were included. The changes that were not included in the model were determined to have no influence on the identified collapse mechanism except to make it more severe and even more certain.

The dead and superimposed dead loads applied to area elements in the global model are summarized in Table 3.1 based on the loading schedule on Sheet S-24 (TISHMAN014741) of the Cantor structural drawings. The weight of steel framing was not included in the dead loads because this load was automatically determined by the analysis program. No additional load was added to account for the weight of connections. The superimposed dead loads listed in the table include fill, finish, ceiling, ductwork, partitions, flooring, beam encasement and fire proofing. The fire proofing load was estimated to be 1.5 psf.

## 3.2.2 Floor Design Live Load

The floor design live loads used in the global collapse analyses were largely based upon the loading schedule shown on Sheet S-24 (TISHMAN014741) of the Cantor structural drawings. A set of calculations from the Office of Irwin G Cantor dated July 1984 supplements the loads with additional live loads for specific floor areas of the upper floors and penthouses (CANTOR0003517-0003762). Table 3.2 summarizes the primary live load used for each level based on the Cantor drawings. In identifying the column bracing design code violations, the live loads on the columns were reduced in accordance with Section C26-903.2 of the Building Code of the City of New York (Ref 8).

Table 3.1 Floor Dead and Superimposed Dead Loads

FLOOR LEVEL	CONCRETE FLOOR	SUPERIMPOSED DEAD	
FLOOR LEVEL	SLAB DEAD LOAD	LOAD	
Penthouse	50 psf	9.5 psf	
Roof	50 psf	19.5 psf	
Floor 24 - 46	50 psf	18.5 psf	
Floor 21 – 23	50 psf	33.5 psf	
Floor 8 -20	50 psf	18.5 psf	
Floor 7 (North Side)	80 psf	See Note 1	
Floor 7 (South Side)	80 psf	See Note 1	
Floor 6 (Office)	56 psf	See Note 1	
Floor 6 (Switchboard)	62 psf	See Note 1	
Floor 5 (Office)	150 psf	See Note 1	
Floor 5 (Mechanical)	150 psf	See Note 1	
Floor 4	56 psf	See Note 1	
Floor 3 (New)	56 psf	See Note 1	

Note 1: Superimposed dead load varies based on extent of concrete beam encasement

Table 3.2 General Floor Live Loads (loading of particular areas not specified)

FLOOR LEVEL	FLOOR LIVE LOAD
Penthouse (old/new)	250/30 psf
Roof	30 psf
Floor 24 - 46	50 psf
Floor 21 – 23	75 psf
Floor 8 -20	50 psf
Floor 7 (North Side)	50 psf
Floor 7 (South Side)	50 psf
Floor 6 (Office)	50 psf
Floor 6 (Switchboard)	100 psf
Floor 5 (Office)	50 psf
Floor 5 (Mechanical)	150 psf
Floor 4	100 psf
Floor 3 (New)	100 psf
Floor 3 (Existing)	100 psf
Floor 2 (New)	150 psf
Floor 2 (Existing)	150 psf
Floor 1(Lobby)	100 psf
Floor 1 (Existing)	225 psf

# 3.2.3 Curtain Wall Superimposed Dead Load

The curtain wall superimposed dead loads applied to all perimeter wind girders in the global model were based upon information contained in a set of calculations from the Office of Irwin G Cantor dated July 1984 (CANTOR0003517-0003762). In this document the North and South façade curtain wall loads were listed as 25 psf, and the East and West façade curtain wall loads were listed as 30 psf. According to the Emery Roth architectural drawings, the curtain wall system consisted of 1" thick glass, granite, 18 gauge galvanized sheets and gypsum board.

#### 3.2.4 Load Combinations

A single load combination was used in all collapse analyses to represent the effect of sustained gravity loads present on the structure at the time of collapse. This load combination is as follows:

100% Dead Load + 100% Superimposed Dead Load + 25% Design Live Load

In each analysis, the sustained load on a member based on the above load combination was compared to the capacity of the member based on the material strength and structural capacity assumptions described in Section 3.4.

# 3.2.5 Additional Loading Considerations

Except for the northeast corner of the building, where a portion of the floor slab was determined to remain following the floor failure mechanism explained in Section 5.4.1, all load corresponding to a floor area was removed from the global collapse model when that area was determined to have collapsed. In other words, no assumptions were made regarding partial floor slabs hung from the remaining structure or the accumulation of load at other locations in the building. Although floor areas were unlikely to detach completely from adjacent structure, the use of this theory is conservative and avoids arbitrary speculation.

# 3.3 Consideration of Debris and Fire Damage

# 3.3.1 Debris Damage

The debris damage to the building that is considered likely to have been present prior to its collapse is shown in Figure 3.1. Damage occurred only on the exterior south and west elevations of the structure. The extent of debris damage was determined from a comprehensive analysis of approximately 250 photographs of the WTC7 building prior to collapse (see Appendix F for sources).

This damage was not included in the global collapse model; however, it was evaluated and determined to have no influence on the cause or the character of the progression of global collapse.

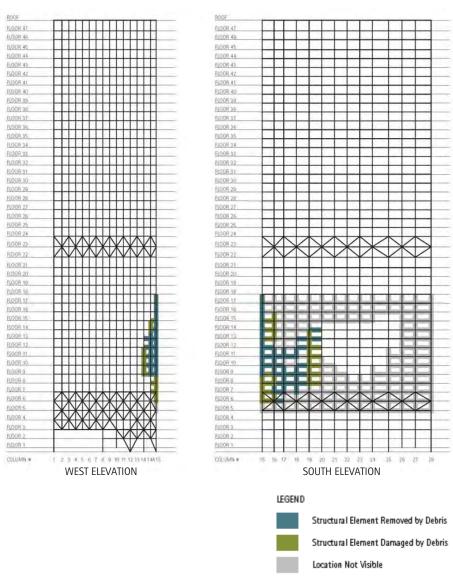


Figure 3.1 Debris damage to WTC7 from visual evidence documented in appendix of report by Dr Fred Mowrer (Ref 14)

# 3.3.2 Fire Damage and Thermal Effects

The traveling office contents fires present on several floors of WTC7 throughout the day on 11 September 2001 and at the time of collapse were determined to have no influence on the cause or the character of the progression of global collapse. Although thermal effects and the presence of fire in the building are critical to the initial local failure mechanism, for the purposes of the global collapse, ambient conditions were assumed because elevated temperatures would only make the structure less resilient.

#### 3.4 Material Strength and Structural Capacities

In the global collapse studies documented in this report, the structural capacities of the steel and concrete members of the WTC7 structure were determined using strength design principles specified by the American Institute of Steel Construction (AISC) (Refs 2 and 3) and the American Concrete Institute (ACI) (Ref 1) respectively; however, no strength reduction factors (\$\phi\$ factors) were used in the analyses. Rather than using lower-bound design strengths to determine the structural capacities of the members, the global collapse analysis was carried out using either material strengths averaged from actual mill and field test report data or expected material strengths obtained by scaling up the lower-bound design values by the appropriate scale factors from Chapters 5 and 6 of ASCE/SEI 41-06 (Ref 5).

This approach is consistent with the methodology prescribed in ASCE/SEI 41-06 for the assessment of existing structures and results in estimates of capacity that are higher than design codes would permit.

## 3.4.1 Material Strengths - Steel

Table 3.3 lists the actual material strengths used in the global collapse analyses for the four primary types of structural steel in the building, averaged from approximately 300 available mill test reports (CANTOR 0013115 – 0013190). Because test reports were unavailable for the concrete slab reinforcement, metal decking, shear studs and high strength bolts, the expected values listed in Table 3.4 were used in the analyses.

Table 3.3 Actual steel strengths averaged from test reports

STEEL GRADE	YIELD	ULTIMATE
STEEL GRADE	STRENGTH	STRENGTH
A572 Grade 50	59.9 ksi	83.1 ksi
A572 Grade 42	50.5 ksi	76.5 ksi
A36	46.0 ksi	73.3 ksi
CSA G40.21-44W	49.6 ksi	76.3 ksi

Table 3.4 Expected steel strengths based on scale factors from ASCE/SEI 41-06

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TYPE OF STEEL	SCALE	YIELD	ULTIMATE		
TIFE OF SIEEL	FACTOR	STRENGTH	STRENGTH		
Plain WWF (A82-70 / A185-70)	1.25	_	87.5 ksi		
Regular steel rebar (A615)	1.25	75 ksi	112.5 ksi		
Metal Decking (A446 / A653)	1.05	-	34.7 ksi		
Headed Shear Studs (A108)	1.5	-	97.5 ksi		
High Strength Bolts (A325F)	1.1	-	132 ksi		
E70XX Electrodes	1.1	-	77 ksi		

# 3.4.2 Material Strengths - Concrete

The strength of the concrete slabs used in the global analyses was calculated as the average of 254 available concrete cylinder test samples for Floors 7 through 13 (see Appendix F for sources). The actual average 28-day compressive strength (f'c) was determined to be 4002 psi.

## 3.4.3 Additional Material Properties

Additional steel and concrete material properties that were used in the floor collapse analyses are provided within the report in Appendix B.

# 3.5 Additional Analysis Details

In addition to the global analysis assumptions listed above, there are several details that deserve particular attention. These details are critical to the performance of the building, its vulnerability to collapse and its behavior during collapse. Therefore, conclusions reached by an analysis that does not take them into account have no factual basis.

#### 3.5.1 Shear Studs on Girders

Although the original structural construction documents issued by Cantor indicate no shear studs on the girders at any level, Guy Nordenson and Associates discovered from the comprehensive document review and cataloguing process that some shear studs were added to many of the girders at all floor levels while the building was under construction. According to the documentation (including CANTOR0008845, CANTOR006189, SHCA0001824, TISHMAN014778-0147780, TISHMAN000315, TISHMAN000300, PANYNJ0095825, CANTOR0016546) the studs were added to increase the vertical load carrying capacity of certain floor areas. Therefore, any additional resistance to disproportionate collapse that was created as a result of this addition was purely coincidental.

#### 3.5.2 Trench Header Ducts

Both the structural and electrical construction documents as well as the Nicholas J Bouras metal deck shop drawings (SOM 0090380 – 0090388, CONEDEXP 0020051, CANTOR2004996 – 2005031) and the MacFab trench duct shop drawings (CANTOR2005508 – 2005521) indicate that trench header ducts existed within the concrete slabs on the majority of the WTC7 floor levels. The trench header ducts were used as part of the electrified cellular floor system within the building. This system utilized the flute cavities under the metal deck to run telephone, power, and signal wiring. In areas where it was necessary to run wiring perpendicular to the direction of the metal deck flutes, trench header ducts were needed. These ducts are typically metal boxes that rest on top of the metal deck within the same depth as the topping slab, disrupting its continuity.

Figure 3.2, the electrical floor plan (PANYNJ0102202), shows the location of the trench header ducts on typical floor levels (Floors 8 to 20). Each red line indicates a 21" to 36"-wide location where concrete was omitted and replaced by a thin-walled metal trench header box (Figure 3.3). The segmentation caused by these trench header ducts reduced the integrity of the diaphragm and had a significant effect on the behavior of the building in its response to fire and during the propagation of collapse.

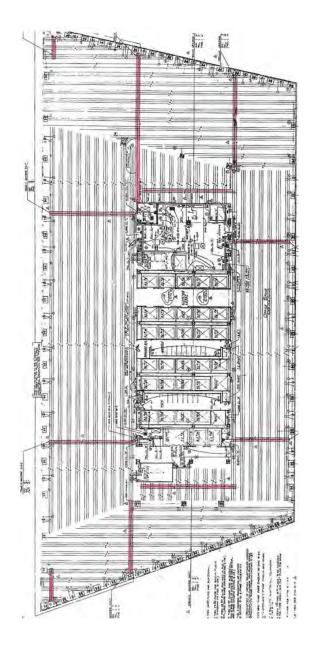


Figure 3.2 Location of trench header ducts (ie concrete slab discontinuities) on typical floors of WTC7 shown in red (PANYNJ0102202)

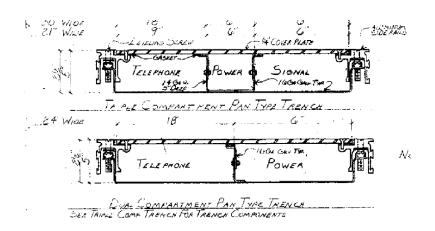


Figure 3.3 Trench header detail from MacFab WTC7 shop drawing T5 (CANTOR2005517)

### 4.0 ASSESSMENT OF STRUCTURAL VULNERABILITY

Both in its overall design and in its details, the WTC7 structure possessed numerous vulnerabilities and design deficiencies as well as one significant structural code violation that made it particularly vulnerable to disproportionate collapse. The source of these vulnerabilities was a general lack of consideration for structural integrity and load path redundancy on the part of the design team. The primary issues are summarized as follows:

- Non-code compliant lateral bracing of columns
- Lack of four-sided lateral support for interior columns
- Multiple interdependent transfer structures
- Discontinuity of concrete slab diaphragm due to trench headers
- Long spans and large tributary areas of interior columns

The code violations and structural vulnerabilities caused the progression of the local floor slab collapse to a global collapse on 11 September 2001 as explained in Section 5.0.

## 4.1 General Structural Vulnerabilities

A well-designed structure should have sufficient structural integrity and redundancy to withstand a local failure, such as the failure of a connection or the loss of a girder, beam or column, with limited repercussions. As a result of the following characteristics, the WTC7 structure was less redundant and robust and therefore less resistant to disproportionate collapse than it should have been.

#### 4.1.1 Large Tributary Areas of Interior Columns

WTC7's east and north floor beam spans of 53 ft and 52 ft respectively were unusually long. The main reason for the increased spans was likely the combination of the project requirement for a large floor plate area and the need to resolve the column layout with the existing structure and program below the office tower. The long floor spans resulted in several interior core columns on the east side of the building supporting particularly large tributary areas. The tributary areas of Columns 76, 79, 80 and 81 were 1470 ft², 1891 ft², 1363 ft² and 1410 ft² respectively (Figure 4.1). Together, these four columns supported approximately 15% of the building's floor plate.

The integrity of these columns consequently played an especially critical role in the integrity of the building as a whole. The loss of just one long-span girder would result in the failure of a large area of floor slab, which would be difficult to arrest at the floor levels below, especially considering the limited amount of reinforcement used in the concrete floor slabs. Similarly, a column failure, such as the failure of Column 79, would result in the loss of almost 5% of the building's floor plate which provides out-of-plane bracing to the perimeter frame.

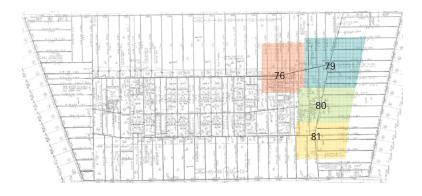


Figure 4.1 Approximate tributary zones of eastern interior columns

#### 4.1.2 Multiple Interdependent Transfer Structures

WTC7 contained numerous transfer structures on its lower floors in order to integrate the new building footprint and column layout of the upper levels with the existing footprint and columns of the Con Edison substation below. These included eight 9'-deep cantilevering plate girders transferring eight north perimeter columns to set-back columns at Floor 7; three transfer trusses between Floors 5 and 7; and several additional transfer girders (Figures 4.2 and 4.3). Transfer structures create interdependency between columns such that if one column fails, it implicates the other columns supported by it. None of the transfer systems were designed with secondary load paths or with higher levels of safety that could have provided robustness or redundancy in the system.

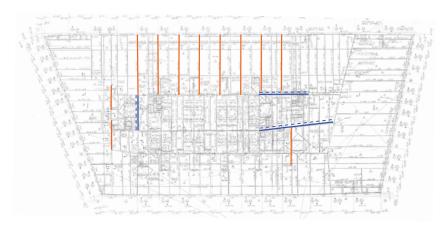


Figure 4.2 Transfer trusses (blue) and Transfer girders (orange) at Floor 7

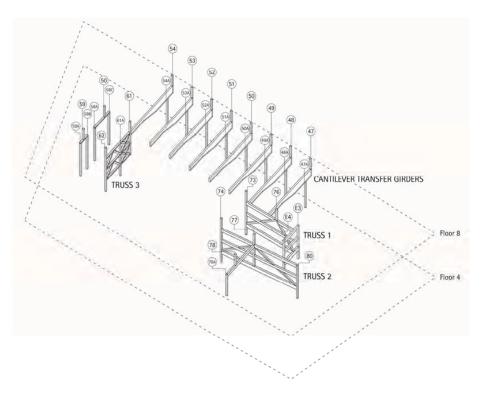


Figure 4.3 Transfer structures between Floors 5 and 7  $\,$ 

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Additionally, in several locations the transfer structures were doubled up such that one transfer structure was supported on another, which created further interrelation and interdependency of the structure as a whole. For example, the transfer girder supporting Column 78 was supported on Transfer Truss 2. Also, the deep transfer girders supporting Columns 47 and 48 were supported on Transfer Truss 1 which in turn was supported by another transfer girder at Floor 5 (Figure 4.4). This configuration created a situation where the failure of Column E3 or E4 would result in the direct loss of vertical support for at least Columns 76, 47 and 48, affecting both the interior and exterior structure of the building.

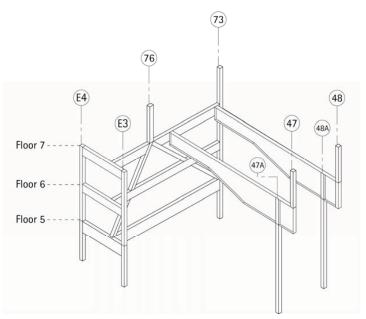


Figure 4.4 Example of interconnected transfer structures in WTC7 (Truss 1)

These types of interdependencies in the structure made it nearly impossible for any type of local collapse to remain local. In this respect, the structure was designed with virtually no consideration of the standards of the time for structural integrity and the prevention of disproportionate collapse, which included provisions in the Building Code of the City of New York (Ref 8) and ANSI's Minimum Design Loads for Buildings and Other Structures (Ref 4).

### 4.1.3 Discontinuity of Concrete Floor Slab due to Trench Headers

As described in Section 3.5, the majority of the building's concrete floor diaphragms were subdivided by trench header ducts, hollow channels in the floor slab where the concrete was removed to allow for the distribution of electrical wires that comprised the electrified cellular floor system. These trench headers disrupted the continuity of the concrete floor slab such that what was intended to be a unified rigid diaphragm was in reality a number of slab segments tied together by only the metal deck and trench header boxes.

A floor diaphragm plays a critical role in connecting all structural components to a building's lateral system. Typically, the frames or walls of a building's lateral system are oriented and distributed in such a way that when they act together, they provide sufficiently stiff and strong lateral resistance to wind and earthquake loads in all directions. However, if the floor slabs are divided into segments by trench headers and no horizontal bracing is added to retain the in-plane resistance of the diaphragm, the components of the lateral force resisting system cannot act together as they were designed. This subdivision could result in a number of problems including insufficient lateral bracing for columns, increased lateral drifts and torsional movement of structural components.

A disruption in the integrity of the floor slab also means that a local floor slab collapse would be less likely to be arrested by the floor below due to its pre-existing segmentation.

### 4.2 Lateral Bracing Code Violation

## 4.2.1 Description of Code Requirement

The design and construction of WTC7 was governed by the Building Code of the City of New York and its referenced standards. Section C26-1001.2 of the Building Code of the City of New York requires that "members used to brace compression members shall be proportioned to resist an axial load of at least 2 percent of the total compressive design stress in the member braced, plus any transverse shear therein" (Ref 8 and Figure 4.5). This code requirement applied to the bracing of all interior and exterior columns in the WTC7 structure. The purpose of this requirement is to ensure that columns and other compression members are capable of developing their expected compression capacity without becoming unstable and buckling. When computing a column's compression capacity, a structural engineer assumes that the column is laterally braced at the floor levels of the building. The code provision provides a straightforward way to compute the lateral forces imposed on the floor framing members and connections as they brace the columns so that the designer may account for the transfer of these forces.

C26-1001.2 Bracing. — Unless otherwise specified in the reference standards, members used to brace compression members shall be proportioned to resist an axial load of at least 2 per cent of the total compressive design stress in the member braced, plus any transverse shear therein.

Figure 4.5 1979 NYC Building Code excerpt regarding lateral bracing (Ref 8)

### 4.2.2 Bracing Member Requirement

Section C26-1001.2 of the Building Code of the City of New York uses the term "members" to describe the components of the structure that are able to provide bracing to compression members. The term can refer to both steel and concrete structural components surrounding a compression element if they are expressly designed and detailed for these forces. Because no positive connection between the steel columns and the concrete floor slab was detailed in the WTC7 structure, the concrete floor slab did not participate in the lateral bracing of the columns. Section 4.2.3 provides additional explanation for the exclusion of concrete in the total lateral bracing capacity.

Because the concrete did not participate in the lateral bracing of interior and exterior columns, the bracing capacity requirement had to be satisfied by the girders and beams framing into these columns. The 2% code provision applies to the sum of the capacities of the steel members bracing a column along each axis, with the capacity in the weakest direction governing (Figure 4.6). For example, if a column has one girder framing into each flange, the lateral bracing provided to the column in its major axis is the sum of the tension capacity of one girder connection and the compression capacity of the other girder connection (Figure 4.7a). The bracing capacity of each girder in tension and compression is generally governed by its connection to the column.

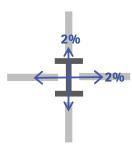


Figure 4.6 Diagram of column lateral bracing requirement in NYC Building Code

If a column does not have paired girders bracing it along each axis, the 2% axial force requirement must be satisfied in tension and compression in each connection (Figure 4.7b). The WTC7 building contained numerous interior columns that were braced on only three sides, including Columns 79 and 80 (see Figure 4.11).

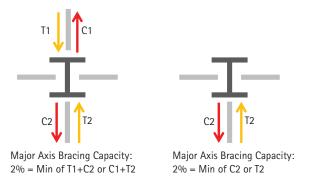


Figure 4.7a,b Four-sided and three-sided lateral bracing comparison (T=tension capacity of girder-to-column connection, C=compression capacity of girder-to-column connection)

### 4.2.3 Role of Concrete in Lateral Bracing

While a concrete slab can be designed to provide lateral bracing to interior columns, it is evident that the concrete floor slabs in the WTC7 building were neither designed to function as lateral bracing nor capable of providing lateral force resistance. In order for a concrete slab to contribute to the lateral bracing of an interior column, it must be detailed such that the steel column is able to bear against it in compression. Because concrete has a tendency to shrink when it dries, even if wet concrete were raked directly against a column, without an embedded steel connection the concrete would pull away from the column surface when drying and no longer provide bearing resistance.

The WTC7 construction documents provide no details to indicate a positive connection between the concrete slab and the columns. Furthermore, the specifications do not provide any direction to the contractor that the concrete slab had to be poured contiguous with the steel columns. Without a specific detail or direction to the contractor provided in the contract documents, it could reasonably be expected that the full-depth concrete slab thickness would not have been maintained around the columns, especially in the difficult-to-access areas adjacent to the column webs between their flanges.

While the WTC7 construction documents provide no slab-to-column detail, they indicate on Sheet S24A that the floor-to-column connection was detailed to allow pipe shafts to run vertically through the slabs adjacent to a number of interior columns (Figure 4.8). The plumbing construction documents specify that Columns 79 and 81, columns with the largest lateral bracing force requirements in the building, had as many as four plumbing lines running through the floor slabs directly adjacent to them. These details confirm that the designers did not intend for the concrete slab to provide lateral bracing to the interior columns.

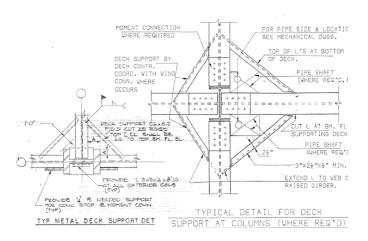


Figure 4.8 Typical detail on Sheet S24A of Cantor construction documents showing allowance for openings in slabs adjacent to interior columns (TISHMAN014742)

In addition to pipe shafts, the concrete floor slab was also interrupted by trench header ducts at most floor levels. A number of these trench headers were located directly adjacent to interior columns, including Column 81. In these locations, the thin walls of the trench headers and bare metal deck would not have been able to adequately brace the columns.

Finally, the WTC7 building was designed with the intention that it would be customized by its tenants at a later date. Structural modifications included girder web penetrations and floor slab removal to create double-height floor space. Had Cantor intended for the concrete to function as a critical element in the lateral bracing of columns, they would not have permitted the removal of floor slabs adjacent to columns.

### 4.2.4 Connection Guidance by Engineer of Record to Contractor

According to the project specifications issued by Cantor as the Engineer of Record, the contractor and its fabricator were responsible for the design and detailing of all connections that were not completely designed and detailed by the Engineer of Record in the construction documents. Because the interior girder-to-column and beam-to-girder connections were not fully designed in the structural construction documents, their design was the responsibility of the contractor.

Although the contractor and its fabricator were responsible for the design and detailing of these connections, Cantor, as the Engineer of Record, was responsible for providing all criteria required for the design. These criteria included all code-based design loads (ie shear and axial force for the pinned girder-to-column connections) as well as any additional requirements or restrictions deemed necessary by the Engineer of Record. Specifically, it was the Engineer of Record's responsibility to determine the axial force requirements for the design of the girder-to-column connections resulting from Section C26-1001.2 of the Building Code of the City of New York.

The General Notes on Sheet S-20 (TISHMAN014737) of the structural construction documents as well as the Typical Shear Connection Detail Notes on Sheet S-24 (TISHMAN014741) provided the shear force design requirements for the interior girder-to-column connections. These notes referred to load tables in AISC for "standard" shear connections. Nowhere in the construction documents or in the specifications was any direction provided as to the axial force design requirements for these connections. It is evident that the design team, including the contractor's fabricator, considered the interior girder-to-column connections to be standard connections for which the standard AISC tables applied when in fact each of these connections should have been specified as a non-standard connection and designed for specific a shear and axial force.

In addition to specifying design shear forces and other criteria for the interior connection design, Cantor provided the contractor with guidance on connection type in the form of typical details in the construction documents. Figure 9 taken from Sheet S-24 shows their typical beam-to-column detail. The detail appears to show two different connection types: a shear tab (or fin) connection and a bolted double angle connection. Within this one detail, there was conflicting guidance to the contractor's fabricator as to which type of connection to use. Furthermore, because the bolts were not called out in the double-angle connection, it is unclear as to whether this connection was meant to represent a header-type connection or a knife-type connection (see Section 2.1.3 for definition of types).

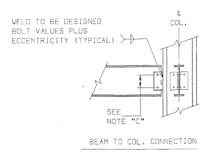


Figure 4.9 Typical beam-to-column connection detail on Sheet S-24 of the structural construction documents showing both shear tab and double angle connections (TISHMAN014741)

#### 4.2.5 Lateral Bracing Capacity of Steel Girder Connections per AISC

As described in Section 2.1, the girder-to-column connections used in WTC7 fall into three categories: seated connections, double-angle header connections (ie angles bolted to the column), and double-angle knife connections (ie angles welded to the column). While the seated and header connections for the most part had sufficient capacity in tension and compression to provide adequate lateral bracing to the columns, the double-angle knife connections had exceptionally low axial capacity in tension.

The welded double-angle shear connections are weak in tension due to the recommended limitations that AISC places on their geometry. These include limits on the welding of the top and bottom edges of the angles to allow for rotational flexibility of the girder (Refs 2 and 3) and constraints on the minimum angle leg length and the maximum angle thickness. These AISC provisions make it virtually impossible to design a welded double-angle connection for a significant tensile force. The connection is not only rotationally flexible but also axially flexible. A tension force on the connection will cause the angle legs to bend and pull away from the column face.

The tables in Appendix C provide a summary of the allowable design capacities of the WTC7 girder-to-column connections in tension and compression that were used in the code check described in Section 4.2.7. The geometry and detailing of each connection used to determine its axial capacity was taken from the latest steel shop drawing issued for that particular connection. The capacity of the seated connection type in tension is governed by the shear capacity of the fully-tightened bolts. Table 1.5.2.1 from the 1980 AISC Manual of Steel Construction (Ref 2) was used to compute the allowable tensile capacity. Its capacity in compression is governed by the fully-tightened bolt shear capacity according to Table 1.5.2.1 as well. The capacity of the double-angle header connection is governed by prying action on the bolts. The table on Page 4-88 of the 1980 AISC was used to estimate the allowable tensile capacity except where capacities

were not well above the design requirements, and the more detailed procedure on Pages 4–89 to 4–90 was used. Its compression capacity is assumed to be governed by weld failure at the girder web according to AISC Table 1.5.3. Design strengths rather than the expected strengths listed in Section 3.4 of this report were used to compute the design capacities listed in these tables as they are meant to represent the axial capacities of the connections that would have been computed by a structural engineer at the time the building was designed. The effects of vertical shear forces on the axial capacities of the connections were conservatively disregarded in the calculations.

AISC provides no direct guidance on the design of a welded double-angle connection for tension loads. An engineer designing such a connection for tension would typically assess the capacity by assuming that the ends of the angles are rotationally free as shown in Figure 4.10. The failure modes checked by the engineer would therefore include direct tension perpendicular to the axis of the fillet welds and flexural failure in the legs of the angles, with flexural yielding of the angles governing for all angle sizes used in the building (typically L4x3x3/8's). The allowable tension capacities listed for these connections in the tables in Appendix C are based on the bending equations in Section 1.5.1.4 of the 1980 AISC, and the compression capacities are based on fully-tightened bolt shear failure according to 1980 AISC Table 1.5.2.1. Had these calculations been performed by the design team, the team would have determined that the allowable design tension capacity of this type of connection was significantly lower than necessary to function as a proper lateral brace.

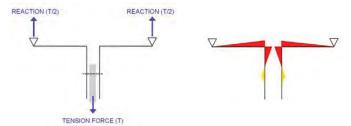


Figure 4.10 Behavior assumed for assessment of tension capacity of welded double-angle connections (free body diagram and bending moments)

In fact, using the western connections to Column 79 as an example, Table 4.1 demonstrates that it is impossible to design a welded double-angle connection for 2% of the design gravity loads in Column 79 below Floor 11, even when optimal angles are used because the bending demands in the angles are too high. For this calculation, L8x4x1 angles (which are significantly larger than the actual angles used in the WTC7 building) were assumed as they were determined to be the most favorable commonly-available rolled angle for bending because they combine a short angle leg with a large thickness. Also, the connection depth was assumed to be the depth of the flat face of the web which is the maximum possible connection depth that could have been used.

Table 4.1 Confirmation of Impossibility to Design Western Welded Double Angle Knife Connection at Column 79 for 2% Bracing Requirement below Floor 11

FLOOR	at Column 79 for 2% Bra 2% TENSION FORCE DESIGN	CONNECTION	MAX TENSION	MAX BRACING
LEVEL	REQUIREMENT BASED ON	TYPE	CAPACITY USING	POSSIBLE (% OF
LEVEL	DESIGN GRAVITY LOADS [KIPS]	TYPE	L8x4x1 [KIPS]	REQUIRED)
Roof	6.8	Header PI		
Floor 47	10.5	Header PI		
Floor 46	14.1	Knife	143.1	1018%
Floor 45	17.7	Knife	143.1	808%
Floor 44	21.7	Knife	143.1	660%
Floor 43	-	-	-	-
Floor 42	25.8	Knife	143.1	556%
Floor 41	-	-	-	-
Floor 40	29.5	Knife	143.1	485%
Floor 39	33.2	Knife	143.1	431%
Floor 38	36.9	Knife	143.1	388%
Floor 37	40.6	Knife	143.1	353%
Floor 36	44.3	Knife	143.1	323%
Floor 35	48.0	Knife	143.1	298%
Floor 34	51.7	Knife	143.1	277%
Floor 33	55.4	Knife	143.1	258%
Floor 32	59.1	Knife	143.1	242%
Floor 31	62.8	Knife	143.1	228%
Floor 30	66.6	Knife	143.1	215%
Floor 29	70.3	Knife	143.1	203%
Floor 28	74.1	Knife	143.1	193%
Floor 27	77.8	Knife	143.1	184%
Floor 26	81.6	Knife	143.1	175%
Floor 25	85.4	Knife	143.1	168%
Floor 24	89.2	Knife	143.1	160%
Floor 23	93.9	Knife	143.1	152%
Floor 22	98.7	Knife	143.1	145%
Floor 21	103.5	Knife	143.1	138%
Floor 20	107.3	Knife	143.1	133%
Floor 19	111.2	Knife	143.1	129%
Floor 18	115.0	Knife	143.1	124%
Floor 17	118.9	Knife	143.1	120%
Floor 16	122.8	Knife	143.1	117%
Floor 15	126.7	Knife	143.1	113%
Floor 14	130.6	Knife	143.1	110%
Floor 13	134.5	Knife	143.1	106%
Floor 12	138.4	Knife	143.1	103%
Floor 11	142.3	Knife	143.1	101%
Floor 10	146.2	Knife	143.1	98%
Floor 9	150.2	Knife	143.1	95%
Floor 8	154.1	Knife	143.1	93%
Floor 7	159.2	Knife	94.7	59%
Floor 6	162.1	Knife	82.4	51%
Floor 5	170.0	Knife	131.9	78%
Floor 4	175.9	Knife	118.7	67%
Floor 3	180.5	-	-	-
Floor 2	184.4	-	-	-

### 4.2.6 Influence of Four-Sided Bracing

Tensile capacity in girder-to-column connections that brace interior columns is especially critical for columns that are not braced on all four sides. Four-sided bracing by girders that are composite with the concrete slab can generally provide adequate lateral force resistance through compression on the girders on two orthogonal sides of the column. The unpaired connection in a column braced on only three sides, however, must provide lateral force resistance in both tension and compression if the concrete slab is not detailed to provide direct bracing capacity.

The structural framing plan for typical floors illustrates that twelve of the twenty-four interior columns used in the WTC7 structure were braced on only three sides, and therefore their unpaired connections were subject to the 2% force requirement in both tension and compression (Figure 4.11). Among these twelve three-sided cases on a typical floor level, ten of the unpaired connections were welded double-angle knife connections that were extremely weak in tension including Columns 79 and 80.

Prior to floor failure on 11 September 2001, tenuous stability of these vulnerable columns likely resulted from a few inadvertent, indirect load paths through which the surrounding intact floor slab permitted bracing of the columns, even though the slab itself did not directly brace them. In addition to the direct bracing provided by the girder-to-column connections, which should have been designed to provide the full bracing capacity requirement, possible accidental bracing mechanisms may have included:

- Resistance from the orthogonal girders framing into the column (Action 1): horizontal shear transfer from the column to the girder web through the connection causes the girder to bend as a cantilever back to the closest floor beam framing into the girder (Figure 4.12)
- Resistance from the orthogonal girders framing into the column (Action 2): shear horizontal transfer from the column to the girder web through the connection induces torsion in the girder to transfer shear to the top flange of the girder and into the concrete slab through the shear studs (Figure 4.13)

Because these accidental column bracing mechanisms were activated by the presence of a surrounding intact concrete slab, when the slab was lost on one side of a column, these secondary load paths also disappeared.

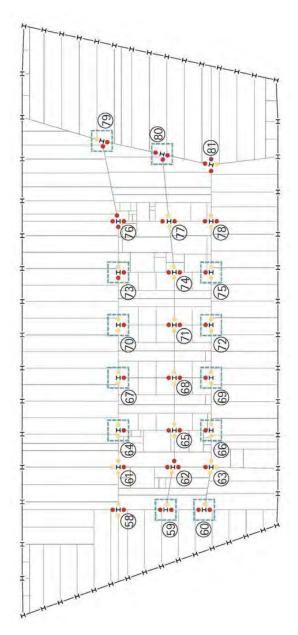


Figure 4.11 Typical WTC7 floor framing plan (red dots indicate knife connections which are vulnerable in tension; yellow dots indicate seated or header connections; blue boxes indicate interior columns with three-sided bracing)

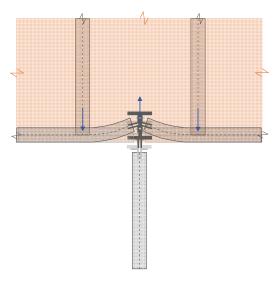


Figure 4.12 Diagram of accidental lateral bracing provided by minor axis bending of orthogonal girders when floor beams are present to provide reactions (configuration similar to Column 70 at core shown, deformations exaggerated)

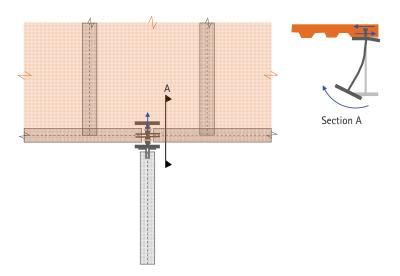


Figure 4.13 Diagram of inadvertent lateral bracing provided by torsional behavior of orthogonal girders when slab is intact (configuration similar to Column 70 at core shown, deformations exaggerated)

## 4.2.7 Violation of Code Requirement

As a result of the extensive use of welded double angle girder-to-column connections and three-sided interior column bracing as well as the lack of connection between the interior columns and concrete slab, over 46% percent of the floor-to-interior column joints in the building did not meet the 2% code requirement in at least one direction.

Tables 4.2 through 4.25 summarize the results of the lateral bracing design code check for all interior columns in the building. All rows highlighted in orange represent lateral bracing design code violations. The tables in Appendix C provide additional details regarding the axial capacity of each connection providing lateral bracing. In Tables 4.2 through 4.25, "No info" refers to a girder-to-column connection for which no information was available. "NC" refers to a girder-to-column connection whose capacity was not checked. These connections were typically large gusset plate connections to interior braced frames or trusses, so it can be reasonably assumed that they had sufficient axial capacity to meet the 2% requirement.

The column design loads listed in Tables 4.2 through 4.25 that were used to determine the code bracing force requirements were taken from the SAP2000 global model rather than the column schedule on Sheet S-17 of the structural construction documents. A comparison of the values shows that the values from the SAP2000 model are consistently less than the values in the column schedule; therefore, the code check is conservative and the number of code violations is a lower-bound estimate.

#### 4.2.8 Relevance to Global Collapse

This evaluation of the lateral bracing condition of the WTC7 interior columns demonstrates that even prior to the initiation of local floor slab collapse, the building was in a tenuous state, and many of the interior columns were already vulnerable to buckling. Section 5.0 explains how the inadequate bracing of the interior columns was directly responsible for the progression of global collapse.

Table 4.2 Column 58 Lateral Bracing Code Check (orange indicates code violations)

		Required 2% Bracing to the Column Force North-South Direction Load		,
Floor	Design Compression			
Level	Load in Column			Load in Column
D (	[Kip]	[Kip]	[Kip]	[Kip]
Roof	90	1.8	98.5	143.2
Floor 47	322	6.4	8.1	256.7
Floor 46	455	9.1	157.3	209.8
Floor 45	579	11.6	96.4	136.4
Floor 44	701	14.0	96.4	116.7
Floor 43	825	16.5	96.4	136.4
Floor 42	947	18.9	96.4	116.7
Floor 41	1071	21.4	96.4	136.4
Floor 40	1194	23.9	96.4	116.7
Floor 39	1270	25.4	96.4	136.4
Floor 38	1393	27.9	96.4	116.7
Floor 37	1518	30.4	96.4	136.4
Floor 36	1641	32.8	96.4	116.7
Floor 35	1765	35.3	96.4	136.4
Floor 34	1889	37.8	96.4	116.7
Floor 33	2014	40.3	96.4	136.4
Floor 32	2139	42.8	96.4	116.7
Floor 31	2264	45.3	96.4	136.4
Floor 30	2389	47.8	96.4	116.7
Floor 29	2515	50.3	96.4	136.4
Floor 28	2641	52.8	96.4	116.7
Floor 27	2768	55.4	96.4	136.4
Floor 26	2894	57.9	96.4	116.7
Floor 25	3021	60.4	96.4	136.4
Floor 24	3147	62.9	96.4	116.7
Floor 23	3307	66.1	96.4	136.4
Floor 22	3466	69.3	96.4	116.7
Floor 21	3627	72.5	96.4	136.4
Floor 20	3758	75.2	96.4	116.7
Floor 19	3891	77.8	96.4	136.4
Floor 18	4019	80.4	96.4	116.7
Floor 17	4148	83.0	96.4	136.4
Floor 16	4277	85.5	96.4	116.7
Floor 15	4407	88.1	96.4	136.4
Floor 14	4535	90.7	96.4	116.7
Floor 13	4666	93.3	96.4	136.4
Floor 12	4795	95.9	96.4	116.7
Floor 11	4927	98.5	96.4	136.4
Floor 10	5061	101.2	96.4	116.7
Floor 10 Floor 9				
	5192	103.8	96.4	136.4
Floor 8	5323	106.5	96.4	116.7
Floor 7	-	-	-	-
Floor 6	-	-	-	-
Floor 5	-	-	-	-
Floor 4	-	-	-	
Floor 3	-	-	-	-
Floor 2	-	-	-	-

Table 4.3 Column 59 Lateral Bracing Code Check (orange indicates code violations)

Table 4.3	able 4.3 Column 59 Lateral Bracing Code Check (c				
			Total Bracing Cap	acity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co	lumn	
Level	Load in Column	Force	North-South Direction	East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	94	1.9	71.1	13.8	
Floor 47	149	3.0	71.1	11.0	
Floor 46	205	4.1	71.1	11.0	
Floor 45	262	5.2	71.1	11.0	
Floor 44	318	6.4	71.1	11.0	
Floor 43	375	7.5	71.1	No info	
Floor 42	431	8.6	71.1	11.0	
Floor 41	487	9.7	71.1	No info	
Floor 40	544	10.9	71.1	11.0	
Floor 39	601	12.0	71.1	No info	
Floor 38	657	13.1	71.1	11.0	
Floor 37	714	14.3	71.1	11.0	
Floor 36	771	15.4	71.1	11.0	
Floor 35	828	16.6	71.1	11.0	
Floor 34	885	17.7	71.1	11.0	
Floor 33	942	18.8	71.1	11.0	
Floor 32	999	20.0	71.1	11.0	
Floor 31	1056	21.1	71.1	11.0	
Floor 30	1114	22.3	71.1	11.0	
Floor 29	1171	23.4	71.1	11.0	
Floor 28	1229	24.6	71.1	11.0	
Floor 27	1287	25.7	71.1	11.0	
Floor 26	1345	26.9	71.1	11.0	
Floor 25	1403	28.1	71.1	11.0	
Floor 24	1462	29.2	71.1	11.0	
Floor 23	1536	30.7	71.1	11.0	
Floor 22	1609	32.2	71.1	11.0	
Floor 21	1684	33.7	71.1	11.0	
Floor 20	1743	34.9	71.1	11.0	
Floor 19	1802	36.0	71.1	11.0	
Floor 18	1861	37.2	71.1	11.0	
Floor 17	1921	38.4	71.1	11.0	
Floor 16	1980	39.6	71.1	11.0	
Floor 15	2040		71.1	11.0	
Floor 15 Floor 14	2040	40.8 42.0	71.1	11.0	
Floor 14 Floor 13		42.0		11.0	
Floor 13 Floor 12	2160 2220	43.2 44.4	71.1 71.1	11.0	
-				11.0	
Floor 11 Floor 10	2281	45.6	71.1		
	2343	46.9	71.1	11.0	
Floor 9	2404	48.1	71.1	11.0	
Floor 8	2465	49.3	71.1	11.0	
Floor 7	-	-	-	-	
Floor 6	-	-	-	-	
Floor 5	-	-	-	-	
Floor 4	-	-	-	-	
Floor 3	-	-	-	-	
Floor 2	-	-	-	-	

Table 4.4 Column 60 Lateral Bracing Code Check (orange indicates code violations)

Table 4.4	able 4.4 Column 60 Lateral Bracing Code Check (c				
			Total Bracing Cap	acity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co	lumn	
Level	Load in Column	Force	North-South Direction	East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	23	0.5	71.1	136.0	
Floor 47	75	1.5	71.1	10.7	
Floor 46	126	2.5	71.1	16.3	
Floor 45	185	3.7	71.1	10.7	
Floor 44	245	4.9	71.1	10.7	
Floor 43	304	6.1	71.1	10.7	
Floor 42	363	7.3	71.1	10.7	
Floor 41	423	8.5	71.1	10.7	
Floor 40	482	9.6	71.1	10.7	
Floor 39	542	10.8	71.1	10.7	
Floor 38	602	12.0	71.1	10.7	
Floor 37	662	13.2	71.1	10.7	
Floor 36	723	14.5	71.1	10.7	
Floor 35	784	15.7	71.1	10.7	
Floor 34	845	16.9	71.1	10.7	
Floor 33	905	18.1	71.1	10.7	
Floor 32	966	19.3	71.1	10.7	
Floor 31	1028	20.6	71.1	10.7	
Floor 30	1089	21.8	71.1	10.7	
Floor 29	1151	23.0	71.1	10.7	
Floor 28	1213	24.3	71.1	10.7	
Floor 27	1275	25.5	71.1	10.7	
Floor 26	1337	26.7	71.1	10.7	
Floor 25	1398	28.0	71.1	10.7	
Floor 24	1461	29.2	71.1	10.7	
Floor 23	1540	30.8	71.1	10.7	
Floor 22	1619	32.4	71.1	10.7	
Floor 21	1699	34.0	71.1	10.7	
Floor 20	1763	35.3	71.1	10.7	
Floor 19	1827	36.5	71.1	10.7	
Floor 18	1891	37.8	71.1	10.7	
Floor 17	1954	39.1	71.1	10.7	
Floor 16	2018	40.4	71.1	10.7	
Floor 15	2083	41.7	71.1	10.7	
Floor 14	2147	42.9	71.1	10.7	
Floor 13	2212	44.2	71.1	10.7	
Floor 12	2277	45.5	71.1	10.7	
Floor 11	2342	46.8	71.1	10.7	
Floor 10	2408	48.2	71.1	10.7	
Floor 9	2473	49.5	71.1	10.7	
Floor 8	2539	50.8	71.1	10.7	
Floor 7	2678	53.6	75.4	11.6	
Floor 6	2678	53.6 54.4	75.4 92.1	112.2	
Floor 5 Floor 4	2883	57.7	116.2 94.3	69.5	
	2988	59.8		No info	
Floor 3	3174	63.5	166.1	40.4	
Floor 2	3257	65.1	96.7	25.0	

Table 4.5 Column 61 Lateral Bracing Code Check (orange indicates code violations)

Table 4.5  Column 61 Lateral Bracing Code Check (ora		orange indicates code violations)		
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	213	4.3	92.1	No info
Floor 47	353	7.1	71.1	214.2
Floor 46	459	9.2	96.4	301.4
Floor 45	543	10.9	50.1	158.7
Floor 44	626	12.5	50.1	138.9
Floor 43	710	14.2	50.1	158.7
Floor 42	792	15.8	50.1	138.9
Floor 41	875	17.5	50.1	158.7
Floor 40	958	19.2	50.1	138.9
Floor 39	1041	20.8	50.1	158.7
Floor 38	1123	22.5	50.1	138.9
Floor 37	1206	24.1	50.1	158.7
Floor 36	1288	25.8	50.1	138.9
Floor 35	1371	27.4	50.1	158.7
Floor 34	1454	29.1	50.1	138.9
Floor 33	1538	30.8	50.1	158.7
Floor 32	1621	32.4	50.1	138.9
Floor 31	1705	34.1	50.1	158.7
Floor 30	1788	35.8	50.1	138.9
Floor 29	1872	37.4	50.1	158.7
Floor 28	1956	39.1	50.1	138.9
Floor 27	2040	40.8	50.1	158.7
Floor 26	2124	42.5	50.1	138.9
Floor 25	2208	44.2	50.1	158.7
Floor 24	2292	45.8	50.1	138.9
Floor 23	2397	47.9	50.1	158.7
Floor 22	2501	50.0	50.1	138.9
Floor 21	2607	52.1	50.1	158.7
Floor 20	2690	53.8	50.1	138.9
Floor 19	2774	55.5	50.1	158.7
Floor 18	2859	57.2	50.1	138.9
Floor 17	2945	58.9	50.1	158.7
Floor 16	3030	60.6	50.1	138.9
Floor 15	3116	62.3	50.1	158.7
Floor 14	3201	64.0	50.1	138.9
Floor 13	3288	65.8	50.1	158.7
Floor 12	3373	67.5	50.1	138.9
Floor 11	3459	69.2	50.1	158.7
Floor 10	3543	70.9	50.1	138.9
Floor 9	3629	70.9	50.1	158.7
Floor 8	3715	74.3	50.1	138.9
Floor 7			NC	
Floor 6	2349 1001	47.0	NC NC	208.1
	959	20.0	NC NC	+
Floor 5 Floor 4	1084	19.2		223.1
	+	21.7	73.3	222.9
Floor 3	-	-	-	-
Floor 2	-	-	-	-

Table 4.6 Column 62 Lateral Bracing Code Check (no code violations)

able 4.6 Column 62 Lateral Bracing Code Check (r		(no code violations)		
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	239	4.8	167.8	199.6
Floor 47	208	4.2	73.3	121.4
Floor 46	256	5.1	71.1	96.2
Floor 45	299	6.0	71.1	100.4
Floor 44	342	6.8	71.1	100.4
Floor 43	385	7.7	71.1	No info
Floor 42	428	8.6	71.1	100.4
Floor 41	471	9.4	71.1	No info
Floor 40	513	10.3	71.1	100.4
Floor 39	556	11.1	71.1	No info
Floor 38	599	12.0	71.1	100.4
Floor 37	641	12.8	71.1	100.4
Floor 36	684	13.7	71.1	100.4
Floor 35	726	14.5	71.1	100.4
Floor 34	769	15.4	71.1	100.4
Floor 33	812	16.2	71.1	100.4
Floor 32	855	17.1	71.1	100.4
Floor 31	898	18.0	71.1	100.4
Floor 30	941	18.8	71.1	100.4
Floor 29	985	19.7	71.1	100.4
Floor 28	1028	20.6	71.1	100.4
Floor 27	1071	21.4	71.1	100.4
Floor 26	1115	22.3	71.1	100.4
Floor 25	1159	23.2	71.1	100.4
Floor 24	1203	24.1	71.1	100.4
Floor 23	1299	26.0	71.1	100.4
Floor 22	1354	27.1	71.1	100.4
Floor 21	1409	28.2	71.1	100.4
Floor 20	1452	29.0	71.1	100.4
Floor 19	1497	29.9	71.1	100.4
Floor 18	1541	30.8	71.1	100.4
Floor 17	1586	31.7	71.1	100.4
Floor 16	1631	32.6	71.1	100.4
Floor 15	1676	33.5	71.1	100.4
Floor 14	1721	34.4	71.1	100.4
Floor 13	1766	35.3	71.1	100.4
Floor 12	1811	36.2	71.1	100.4
Floor 11	1856	37.1	71.1	100.4
Floor 10	1901	38.0	71.1	100.4
Floor 9	1946	38.9	71.1	100.4
Floor 8	1991	39.8	71.1	100.4
Floor 7	2002	40.0	69.0	134.2
Floor 6	1393	27.9	NC	104.7
Floor 5	1058	21.2	NC NC	119.0
Floor 4	1130	22.6	69.0	86.1
Floor 3	1181	23.6	83.0	116.0
Floor 2	1235	24.7	94.6	120.2

Table 4.7 Column 63 Lateral Bracing Code Check (orange indicates code violations)

Table 4.7	Column 63 Lateral B	racing Code Check	(orange indicates cod	le violations)
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	21	0.4	96.2	138.9
Floor 47	63	1.3	NC	147.5
Floor 46	100	2.0	88.0	156.9
Floor 45	157	3.1	48.0	131.5
Floor 44	213	4.3	48.0	131.5
Floor 43	269	5.4	48.0	131.5
Floor 42	325	6.5	48.0	131.5
Floor 41	381	7.6	48.0	131.5
Floor 40	438	8.8	48.0	131.5
Floor 39	494	9.9	48.0	131.5
Floor 38	551	11.0	48.0	131.5
Floor 37	608	12.2	48.0	116.7
Floor 36	665	13.3	48.0	131.5
Floor 35	722	14.4	48.0	131.5
Floor 34	779	15.6	48.0	131.5
Floor 33	837	16.7	48.0	131.5
Floor 32	894	17.9	48.0	131.5
Floor 31	952	19.0	48.0	131.5
Floor 30	1010	20.2	48.0	131.5
Floor 29	1069	21.4	48.0	131.5
Floor 28	1127	22.5	48.0	131.5
Floor 27	1186	23.7	48.0	131.5
Floor 26	1244	24.9	48.0	131.5
Floor 25	1302	26.0	48.0	131.5
Floor 24	1361	27.2	48.0	131.5
Floor 23	1434	28.7	48.0	138.9
Floor 22	1507	30.1	48.0	138.9
Floor 21	1581	31.6	48.0	138.9
Floor 20	1640	32.8	48.0	131.5
Floor 19	1699	34.0	48.0	131.5
Floor 18	1759	35.2	48.0	131.5
Floor 17	1819	36.4	48.0	131.5
Floor 16	1879	37.6	48.0	131.5
Floor 15	1939	38.8	48.0	131.5
Floor 14	1999	40.0	48.0	131.5
Floor 13	2059	41.2	48.0	131.5
Floor 12	2119	42.4	48.0	131.5
Floor 11	2180	43.6	48.0	131.5
Floor 10	2235	44.7	48.0	131.5
Floor 9	2296	45.9	48.0	131.5
Floor 8	2357	47.1	48.0	131.5
Floor 7	2478	49.6	71.1	138.9
Floor 6	2532	50.6	207.6	117.8
Floor 5	2703	54.1	73.3	331.2
Floor 4	2752	55.0	6.0	116.7
Floor 3	2865	57.3	71.5	195.5
Floor 2	2923	58.5	6.4	12.2

Table 4.8 Column 64 Lateral Bracing Code Check (orange indicates code violations)

	Columni of Lateral D	racing code check	(orange indicates cod	ie violations)
			Total Bracing Capacity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	184	3.7	10.3	NC
Floor 47	254	5.1	14.0	232.8
Floor 46	389	7.8	6.0	339.2
Floor 45	472	9.4	6.0	138.9
Floor 44	555	11.1	6.0	138.9
Floor 43	638	12.8	6.0	138.9
Floor 43	719	14.4	6.0	138.9
Floor 41	801	16.0	6.0	138.9
Floor 40				138.9
	883	17.7	6.0	
Floor 39	965	19.3	6.0	138.9
Floor 38	1046	20.9	6.0	138.9
Floor 37	1128	22.6	6.0	138.9
Floor 36	1210	24.2	6.0	138.9
Floor 35	1292	25.8	6.0	138.9
Floor 34	1374	27.5	6.0	138.9
Floor 33	1456	29.1	6.0	No info
Floor 32	1538	30.8	6.0	No info
Floor 31	1620	32.4	6.0	No info
Floor 30	1702	34.0	6.0	No info
Floor 29	1785	35.7	6.0	138.9
Floor 28	1868	37.4	6.0	138.9
Floor 27	1950	39.0	6.0	138.9
Floor 26	2033	40.7	6.0	138.9
Floor 25	2116	42.3	6.0	138.9
Floor 24	2199	44.0	6.0	138.9
Floor 23	2303	46.1	6.0	143.2
Floor 22	2407	48.1	6.0	143.2
Floor 21	2512	50.2	6.0	138.9
Floor 20	2597	51.9	6.0	138.9
Floor 19	2682	53.6	6.0	138.9
Floor 18	2766	55.3	6.0	138.9
Floor 17	2851	57.0	6.0	138.9
Floor 16	2936	58.7	6.0	138.9
Floor 15	3021	60.4	6.0	138.9
Floor 14	3106	62.1	6.0	138.9
Floor 13	3192	63.8	6.0	138.9
Floor 12	3278	65.6	6.0	138.9
Floor 11	3363	67.3	6.0	138.9
Floor 10	3450	69.0	6.0	138.9
Floor 9	3537	70.7	6.0	138.9
Floor 8	3623	72.5	6.0	138.9
Floor 7	3306	66.1	NC	NC
Floor 6	3306	66.1	86.5	NC
Floor 5	3219	64.4	NC	NC
Floor 4	3371	67.4	86.5	NC
Floor 3	-	-	-	-
Floor 2	-	-	-	_

Table 4.9(	Column 65 Lateral B	racing Code Check	(no code violations)		
			Total Bracing Capacity Provided to the Column  North-South Direction East-West Direct		
Floor	Design Compression	Required 2% Bracing			
Level	Load in Column	Force		East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	213	4.3	117.4	197.3	
Floor 47	297	5.9	203.0	160.4	
Floor 46	317	6.3	69.0	143.2	
Floor 45	338	6.8	69.0	138.9	
Floor 44	359	7.2	69.0	138.9	
Floor 43	380	7.6	69.0	138.9	
Floor 42	401	8.0	69.0	138.9	
Floor 41	421	8.4	69.0	138.9	
Floor 40	442	8.8	69.0	138.9	
Floor 39	463	9.3	69.0	138.9	
Floor 38	483	9.7	69.0	138.9	
Floor 37	504	10.1	69.0	138.9	
Floor 36	525	10.5	69.0	138.9	
Floor 35	545	10.9	69.0	No info	
Floor 34	566	11.3	69.0	No info	
Floor 33	587	11.7	69.0	No info	
Floor 32	607	12.1	69.0	No info	
Floor 31	628	12.6	69.0	138.9	
Floor 30	649	13.0	69.0	138.9	
Floor 29	669	13.4	69.0	138.9	
Floor 28	690	13.8	69.0	138.9	
Floor 27	711	14.2	69.0	138.9	
Floor 26	731	14.6	69.0	138.9	
Floor 25	752	15.0	69.0	138.9	
Floor 24	773	15.5	69.0	138.9	
Floor 23	799	16.0	69.0	138.9	
Floor 22	824	16.5	69.0	138.9	
Floor 21	850	17.0	69.0	138.9	
Floor 20	871	17.4	69.0	138.9	
Floor 19	893	17.9	69.0	138.9	
Floor 18	914	18.3	69.0	138.9	
Floor 17	935	18.7	69.0	138.9	
Floor 16	956	19.1	69.0	138.9	
Floor 15	978	19.6	69.0	138.9	
Floor 14	999	20.0	69.0	138.9	
Floor 13	1021	20.4	69.0	138.9	
Floor 12	1042	20.8	69.0	138.9	
Floor 11	1064	21.3	69.0	138.9	
Floor 10	1086	21.7	69.0	138.9	
Floor 9	1107	22.1	69.0	138.9	
Floor 8	1129	22.6	69.0	138.9	
Floor 7	1166	23.3	NC NC	116.7	
Floor 6	1197	23.9	207.6	140.4	
Floor 5	1012	20.2	NC	164.7	
Floor 4	1031	20.6	207.6	116.7	
Floor 3	1578	31.6	NC	208.1	
Floor 2	1606	32.1	321.5	147.5	
11001 2	1000	JZ.1	3∠1.5	1+7.5	

Table 4.10 Column 66 Lateral Bracing Code Check (orange indicates code violations)

lable 4.10	Column 66 Lateral	Bracing Code Check	(orange indicates co	ide violations)
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	32	0.6	8.1	138.9
Floor 47	77	1.5	NC	192.1
Floor 46	119	2.4	6.0	218.6
Floor 45	170	3.4	6.0	116.7
Floor 44	220	4.4	6.0	116.7
Floor 43	270	5.4	6.0	116.7
Floor 42	320	6.4	6.0	116.7
Floor 41	370	7.4	6.0	116.7
Floor 40	421	8.4	6.0	116.7
Floor 39	472	9.4	6.0	116.7
Floor 38	523	10.5	6.0	116.7
Floor 37	575	11.5	6.0	116.7
Floor 36	625	12.5	6.0	116.7
Floor 35	676	13.5	6.0	116.7
Floor 34	727	14.5	6.0	116.7
Floor 33	779	15.6	6.0	116.7
Floor 32	830	16.6	6.0	116.7
Floor 31	882	17.6	6.0	116.7
Floor 30	933	18.7	6.0	116.7
Floor 29	985	19.7	6.0	116.7
Floor 28	1037	20.7	6.0	116.7
Floor 27	1089	21.8	6.0	116.7
Floor 26	1140	22.8	6.0	116.7
Floor 25	1191	23.8	6.0	116.7
Floor 24	1243	24.9	6.0	116.7
Floor 23	1307	26.1	6.0	138.9
Floor 22	1371	27.4	6.0	138.9
Floor 21	1436	28.7	6.0	138.9
Floor 20	1489	29.8	6.0	116.7
Floor 19	1541	30.8	6.0	116.7
Floor 18	1594	31.9	6.0	116.7
Floor 17	1646	32.9	6.0	116.7
Floor 16	1699	34.0	6.0	116.7
Floor 15	1752	35.0	6.0	116.7
Floor 14	1805	36.1	6.0	112.4
Floor 13	1858	37.2	6.0	116.7
Floor 12	1911	38.2	6.0	116.7
Floor 11	1964	39.3	6.0	116.7
Floor 10	2023	40.5	6.0	116.7
Floor 9	2077	41.5	6.0	116.7
Floor 8	2130	42.6	6.0	116.7
Floor 7	2020	40.4	NC	NC
Floor 6	2020	40.4	224.9	NC NC
Floor 5	1718	34.4	NC	NC NC
Floor 4	1718	34.4	121.1	NC NC
Floor 3	2120	42.4	NC	NC NC
Floor 2	2390	47.8	200.4	NC

Table 4.11 Column 67 Lateral Bracing Code Check (orange indicates code violations)

Table 4.11	Column 67 Lateral	al Bracing Code Check (orange indicates code violations)			
			Total Bracing Cap	acity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co	lumn	
Level	Load in Column	Force	North-South Direction	East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	118	2.4	6.0	186.9	
Floor 47	184	3.7	14.0	174.1	
Floor 46	356	7.1	6.0	401.8	
Floor 45	466	9.3	10.3	196.3	
Floor 44	569	11.4	6.0	161.2	
Floor 43	653	13.1	6.0	183.5	
Floor 42	756	15.1	6.0	161.2	
Floor 41	845	16.9	6.0	183.5	
Floor 40	947	18.9	6.0	161.2	
Floor 39	1050	21.0	6.0	183.5	
Floor 38	1153	23.1	6.0	161.2	
Floor 37	1269	25.4	42.0	365.5	
Floor 36	1360	27.2	6.0	138.9	
Floor 35	1452	29.0	6.0	197.4	
Floor 34	1543	30.9	6.0	138.9	
Floor 33	1635	32.7	6.0	No info	
Floor 32	1727	34.5	6.0	No info	
Floor 31	1822	36.4	6.0	No info	
Floor 30	1918	38.4	12.6	No info	
Floor 29	2011	40.2	6.0	197.4	
Floor 28	2103	42.1	6.0	138.9	
Floor 27	2196	43.9	6.0	197.4	
Floor 26	2288	45.8	6.0	138.9	
Floor 25	2382	47.6	6.0	197.4	
Floor 24	2475	49.5	6.0	138.9	
Floor 23	2592	51.8	6.0	229.5	
Floor 22	2709	54.2	6.0	165.5	
Floor 21	2826	56.5	6.0	225.2	
Floor 20	2920	58.4	6.0	138.9	
Floor 19	3015	60.3	6.0	197.4	
Floor 18	3110	62.2	6.0	138.9	
Floor 17	3205	64.1	6.0	197.4	
Floor 16	3300	66.0	6.0	138.9	
Floor 15	3396	67.9	6.0	197.4	
Floor 14	3491	69.8	6.0	138.9	
Floor 13	3587	71.7	6.0	197.4	
Floor 12	3682	73.6	6.0	138.9	
Floor 11	3778	75.6	6.0	197.4	
Floor 10	3875	77.5	6.0	138.9	
Floor 9	3972	79.4	6.0	197.4	
Floor 8	4068	81.4	6.0	NC	
Floor 7	3664	73.3	NC	NC	
Floor 6	3610	72.2	121.1	NC	
Floor 5	3458	69.2	NC	NC	
Floor 4	3643	72.9	86.5	NC	
Floor 3	-	-	-	-	
Floor 2	-	-	-	-	

1able 4.12	Column 68 Lateral	Bracing Code Check	(no code violations)		
			Total Bracing Capacity Provided to the Column		
Floor	Design Compression	Required 2% Bracing			
Level	Load in Column	Force	North-South Direction	East-West Direction	
5 (	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	141	2.8	69.0	138.9	
Floor 47	238	4.8	140.0	169.8	
Floor 46	303	6.1	69.0	165.5	
Floor 45	365	7.3	69.0	151.8	
Floor 44	414	8.3	69.0	138.9	
Floor 43	465	9.3	69.0	143.2	
Floor 42	513	10.3	69.0	138.9	
Floor 41	564	11.3	69.0	143.2	
Floor 40	612	12.2	69.0	138.9	
Floor 39	663	13.3	69.0	143.2	
Floor 38	711	14.2	69.0	138.9	
Floor 37	774	15.5	48.0	164.7	
Floor 36	816	16.3	69.0	138.9	
Floor 35	860	17.2	69.0	No info	
Floor 34	902	18.0	69.0	No info	
Floor 33	946	18.9	69.0	No info	
Floor 32	988	19.8	69.0	No info	
Floor 31	1039	20.8	69.0	151.8	
Floor 30	1094	21.9	201.6	164.7	
Floor 29	1126	22.5	69.0	152.5	
Floor 28	1156	23.1	69.0	138.9	
Floor 27	1188	23.8	69.0	152.5	
Floor 26	1218	24.4	69.0	138.9	
Floor 25	1249	25.0	69.0	152.5	
Floor 24	1280	25.6	69.0	138.9	
Floor 23	1319	26.4	69.0	152.5	
Floor 22	1356	27.1	69.0	138.9	
Floor 21	1395	27.9	69.0	152.5	
Floor 20	1425	28.5	69.0	138.9	
Floor 19	1457	29.1	69.0	152.5	
Floor 18	1488	29.8	69.0	138.9	
Floor 17	1520	30.4	69.0	152.5	
Floor 16	1551	31.0	69.0	138.9	
Floor 15	1583	31.7	69.0	152.5	
Floor 14	1614	32.3	69.0	112.4	
Floor 13	1646	32.9	69.0	152.5	
Floor 12	1677	33.5	69.0	138.9	
Floor 11	1710	34.2	69.0	152.5	
Floor 10	1741	34.8	69.0	138.9	
Floor 9	1774	35.5	69.0	152.5	
Floor 8	1806	36.1	69.0	138.9	
Floor 7	1782	35.6	NC NC	121.0	
Floor 6	1824	36.5	207.6	147.8	
Floor 5	1425	28.5	NC	298.3	
Floor 4	1436	28.7	207.6	112.4	
Floor 3	1958	39.2	NC	245.2	
Floor 2	1987	39.7	321.5	192.1	

Table 4.13 Column 69 Lateral Bracing Code Check (orange indicates code violations)

Floor	Design Compression	Required 2% Bracing	Total Bracing Cap to the Co	,
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	42	0.8	6.0	138.9
Floor 47	84	1.7	12.4	169.8
Floor 46	136	2.7	6.0	222.9
Floor 45	208	4.2	6.0	143.2
Floor 44	275	5.5	6.0	138.9
Floor 43	331	6.6	6.0	No info
Floor 42	398	8.0	6.0	No info
Floor 41	458	9.2	6.0	No info
Floor 40	525	10.5	6.0	No info
Floor 39	593	11.9	6.0	No info
Floor 38	660	13.2	6.0	138.9
Floor 37	728	14.6	6.0	138.9
Floor 36	795	15.9	6.0	138.9
Floor 35	863	17.3	6.0	138.9
Floor 34	931	18.6	6.0	138.9
Floor 33	1000	20.0	6.0	138.9
Floor 32	1068	21.4	6.0	138.9
Floor 31	1141	22.8	6.0	138.9
Floor 30	1216	24.3	12.6	156.1
Floor 29	1278	25.6	6.0	138.9
Floor 28	1340	26.8	6.0	138.9
Floor 27	1402	28.0	6.0	138.9
Floor 26	1463	29.3	6.0	138.9
Floor 25	1525	30.5	6.0	138.9
Floor 24	1587	31.7	6.0	138.9
Floor 23	1664	33.3	6.0	138.9
Floor 22	1742	34.8	6.0	138.9
Floor 21	1820	36.4	6.0	138.9
Floor 20	1882	37.6	6.0	138.9
Floor 19	1944	38.9	6.0	138.9
Floor 18	2006	40.1	6.0	138.9
Floor 17	2069	41.4	6.0	138.9
Floor 16	2131	42.6	6.0	138.9
Floor 15	2194	43.9	6.0	138.9
Floor 14	2257	45.1	6.0	112.4
Floor 13	2321	46.4	6.0	138.9
Floor 12	2384	47.7	6.0	138.9
Floor 11	2447	48.9	6.0	138.9
Floor 10	2511	50.2	6.0	138.9
Floor 9	2574	51.5	6.0	138.9
Floor 8	2638	52.8	6.0	138.9
Floor 7	2452	49.0	NC	NC
Floor 6	2574	51.5	121.1	NC
Floor 5	2127	42.5	NC NC	NC
Floor 4	2028	40.6	121.1	NC
Floor 3	2677	53.5	NC	NC
Floor 2	3250	65.0	121.1	NC

Table 4.14 Column 70 Lateral Bracing Code Check (orange indicates code violations)

Table 4.14	Column 70 Lateral	Bracing Code Check	: (orange indicates co	de violations)
			Total Bracing Capacity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	80	1.6	6.0	169.8
Floor 47	124	2.5	10.3	165.5
Floor 46	357	7.1	12.4	384.7
Floor 45	499	10.0	12.4	174.1
Floor 44	627	12.5	12.4	174.1
Floor 43	670	13.4	12.4	25.0
Floor 42	813	16.3	12.4	174.1
Floor 41	863	17.3	12.4	25.0
Floor 40	1014	20.3	12.4	174.1
Floor 39	1134	22.7	12.4	196.3
Floor 38	1255	25.1	12.4	174.1
Floor 37	1388	27.8	10.6	269.0
Floor 36	1494	29.9	10.3	151.8
Floor 35	1601	32.0	10.3	210.3
Floor 34	1707	34.1	10.3	151.8
Floor 33	1815	36.3	10.3	210.3
Floor 32	1921	38.4	10.3	151.8
Floor 31	2031	40.6	10.3	225.5
Floor 30	2141	42.8	10.3	174.1
Floor 29	2249	45.0	10.3	210.3
Floor 28	2358	47.2	10.3	151.8
Floor 27	2467	49.3	10.3	210.3
Floor 26	2577	51.5	10.3	151.8
Floor 25	2687	53.7	10.3	210.3
Floor 24	2795	55.9	10.3	147.5
Floor 23	2941	58.8	8.1	203.3
Floor 22	3085	61.7	6.0	165.5
Floor 21	3230	64.6	6.0	181.0
Floor 20	3344	66.9	6.0	143.2
Floor 19	3458	69.2	6.0	174.8
Floor 18	3573	71.5	6.0	143.2
Floor 17	3688	73.8	6.0	174.8
Floor 16	3809	76.2	6.0	147.5
Floor 15	3939	78.8	42.0	72.0
Floor 14	4040	80.8	6.0	138.9
Floor 13	4143	82.9	6.0	152.5
Floor 12	4245	84.9	6.0	138.9
Floor 11	4349	87.0	6.0	152.5
Floor 10	4452	89.0	6.0	138.9
Floor 9	4556	91.1	6.0	152.5
Floor 8	4659	93.2	6.0	138.9
Floor 7	4270	85.4	NC	795.8
Floor 6	4214	84.3	86.5	NC
Floor 5	4104	82.1	NC	NC
Floor 4	4318	86.4	86.5	NC
Floor 3	-	-	-	-
Floor 2	-	-	-	-

Table 4.15 Column 71 Lateral Bracing Code Check (no code violations)

Floor Level Roof Floor 47 Floor 46	Design Compression Load in Column	Required 2% Bracing	Total Bracing Cap		
Roof Floor 47	,	Required 2% Bracing	4 - 41 - 0 -		
Roof Floor 47	Load in Column			Column	
Floor 47		Force	North-South Direction	East-West Direction	
Floor 47	[Kip]	[Kip]	[Kip]	[Kip]	
	105	2.1	69.0	138.9	
Floor 46	170	3.4	94.3	178.4	
F1001 40	254	5.1	96.4	169.8	
Floor 45	334	6.7	75.4	174.1	
Floor 44	398	8.0	75.4	143.2	
Floor 43	436	8.7	75.4	165.5	
Floor 42	518	10.4	75.4	143.2	
Floor 41	553	11.1	75.4	165.5	
Floor 40	646	12.9	75.4	143.2	
Floor 39	715	14.3	76.1	165.5	
Floor 38	783	15.7	76.1	143.2	
Floor 37	864	17.3	72.8	186.9	
Floor 36	919	18.4	73.3	143.2	
Floor 35	976	19.5	73.3	174.8	
Floor 34	1032	20.6	73.3	143.2	
Floor 33	1089	21.8	73.3	174.8	
Floor 32	1146	22.9	73.3	143.2	
Floor 31	1209	24.2	73.3	174.1	
Floor 30	1276	25.5	119.0	186.9	
Floor 29	1324	26.5	73.3	174.8	
Floor 28	1372	27.4	73.3	143.2	
Floor 27	1421	28.4	73.3	174.8	
Floor 26	1469	29.4	73.3	143.2	
Floor 25	1518	30.4	73.3	174.8	
Floor 24	1566	31.3	73.3	143.2	
Floor 23	1637	32.7	96.6	223.1	
Floor 22	1690	33.8	69.0	143.2	
Floor 21	1744	34.9	69.0	174.8	
Floor 20	1786	35.7	69.0	138.9	
Floor 19	1831	36.6	69.0	152.5	
Floor 18	1874	37.5	69.0	138.9	
Floor 17	1919	38.4	69.0	152.5	
Floor 16	1968	39.4	69.0	147.5	
Floor 15	2026	40.5	48.0	214.6	
Floor 14	2058	41.2	69.0	116.7	
Floor 13	2089	41.8	69.0	152.5	
Floor 12	2121	42.4	69.0	138.9	
Floor 11	2154	43.1	69.0	152.5	
Floor 10	2185	43.7	69.0	138.9	
Floor 9	2218	44.4	69.0	152.5	
Floor 8	2250	45.0	69.0	138.9	
Floor 7	2175	43.5	NC	143.2	
Floor 6	2213	44.3	207.6	143.2	
Floor 5	1682	33.6	207.6 NC	298.3	
Floor 4	1707		NC 207.6	298.3 111.6	
		34.1			
Floor 3 Floor 2	2232 2262	44.6 45.2	NC 321.5	245.2 147.5	

Table 4.16 Column 72 Lateral Bracing Code Check (orange indicates code violations)

Floor	Design Compression	Required 2% Bracing	Total Bracing Cap to the Co	,
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	39	0.8	6.0	138.9
Floor 47	85	1.7	8.1	178.4
Floor 46	170	3.4	8.1	275.3
Floor 45	253	5.1	6.0	178.4
Floor 44	331	6.6	6.0	156.1
Floor 43	361	7.2	6.0	No info
Floor 42	454	9.1	6.0	No info
Floor 41	493	9.9	6.0	No info
Floor 40	597	11.9	6.0	No info
Floor 39	676	13.5	6.0	156.1
Floor 38	756	15.1	6.0	156.1
Floor 37	836	16.7	6.0	156.1
Floor 36	916	18.3	6.0	156.1
Floor 35	997	19.9	6.0	156.1
Floor 34	1078	21.6	6.0	156.1
Floor 33	1159	23.2	6.0	156.1
Floor 32	1241	24.8	6.0	156.1
Floor 31	1327	26.5	6.0	156.1
Floor 30	1415	28.3	14.0	245.2
Floor 29	1489	29.8	6.0	156.1
Floor 28	1562	31.2	6.0	156.1
Floor 27	1636	32.7	6.0	156.1
Floor 26	1709	34.2	6.0	156.1
Floor 25	1783	35.7	6.0	156.1
Floor 24	1854	37.1	6.0	138.9
Floor 23	1946	38.9	11.2	156.1
Floor 22	2026	40.5	6.0	138.9
Floor 21	2105	42.1	6.0	138.9
Floor 20	2169	43.4	6.0	138.9
Floor 19	2233	44.7	6.0	138.9
Floor 18	2297	45.9	6.0	138.9
Floor 17	2361	47.2	6.0	138.9
Floor 16	2425	48.5	6.0	138.9
Floor 15	2489	49.8	6.0	138.9
Floor 14	2554	51.1	6.0	116.7
Floor 13	2619	52.4	6.0	138.9
Floor 12	2683	53.7	6.0	138.9
Floor 11	2748	55.0	6.0	138.9
Floor 10	2814	56.3	6.0	138.9
Floor 9	2879	57.6	6.0	138.9
Floor 8	2944	58.9	6.0	138.9
Floor 7	2740	54.8	NC	267.0
Floor 6	2856	57.1	121.1	NC NC
Floor 5	2306	46.1	NC	NC
Floor 4	2156	43.1	121.1	NC
Floor 3	2855	57.1	NC	NC
Floor 2	3590	71.8	121.1	NC

Table 4.17 Column 73 Lateral Bracing Code Check (orange indicates code violations)

Table 4.17	Column 73 Lateral	Bracing Code Check	ለ (orange indicates co	ode violations)
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	101	2.0	75.4	165.5
Floor 47	147	2.9	10.3	165.5
Floor 46	325	6.5	12.4	412.6
Floor 45	441	8.8	10.3	196.3
Floor 44	554	11.1	10.3	196.3
Floor 43	555	-	-	-
Floor 42	681	13.6	10.3	196.3
Floor 41	683	-	-	-
Floor 40	816	16.3	10.3	196.3
Floor 39	933	18.7	10.3	196.3
Floor 38	1050	21.0	10.3	196.3
Floor 37	1168	23.4	10.3	196.3
Floor 36	1287	25.7	10.3	196.3
Floor 35	1406	28.1	10.3	196.3
Floor 34	1526	30.5	10.3	196.3
Floor 33	1646	32.9	10.3	196.3
Floor 32	1766	35.3	10.3	196.3
Floor 31	1886	37.7	10.3	196.3
Floor 30	2007	40.1	10.3	196.3
Floor 29	2128	42.6	10.3	196.3
Floor 28	2249	45.0	10.3	196.3
Floor 27	_	47.4	10.3	196.3
	2370		10.3	
Floor 26 Floor 25	2492	49.8		196.3
	2614	52.3 54.8	10.3 12.4	196.3
Floor 24	2740			200.6
Floor 23	2887	57.7	14.5	200.6
Floor 22	3027	60.5	10.3	178.4
Floor 21	3168	63.4	10.3	52.3
Floor 20	3284	65.7	10.3	52.3
Floor 19	3400	68.0	10.3	52.3
Floor 18	3516	70.3	10.3	52.3
Floor 17	3633	72.7	10.3	52.3
Floor 16	3756	75.1	12.4	54.4
Floor 15	3886	77.7	18.7	84.0
Floor 14	3991	79.8	6.0	52.3
Floor 13	4096	81.9	6.0	52.3
Floor 12	4201	84.0	6.0	52.3
Floor 11	4307	86.1	6.0	52.3
Floor 10	4415	88.3	6.0	52.3
Floor 9	4521	90.4	6.0	52.3
Floor 8	4628	92.6	6.0	52.3
Floor 7	4340	86.8	NC	317.7
Floor 6	4352	87.0	338.5	No info
Floor 5	6615	132.3	NC	NC
Floor 4	6796	135.9	86.5	NC
Floor 3	-	-	-	-
Floor 2	-	-	-	-

Table 4.18 Column 74 Lateral Bracing Code Check (orange indicates code violations)

1able 4.18	Column 74 Lateral	Bracing Code Check	(orange indicates co	ide violations)	
			Total Bracing Cap	acity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co		
Level	Load in Column	Force	North-South Direction	East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	119	2.4	71.1	143.2	
Floor 47	190	3.8	73.3	178.4	
Floor 46	259	5.2	75.4	165.5	
Floor 45	329	6.6	73.3	165.5	
Floor 44	395	7.9	73.3	165.5	
Floor 43	397	-	-	-	
Floor 42	475	9.5	73.3	165.5	
Floor 41	477	-	-	-	
Floor 40	562	11.2	73.3	165.5	
Floor 39	634	12.7	73.3	165.5	
Floor 38	706	14.1	73.3	165.5	
Floor 37	779	15.6	73.3	165.5	
Floor 36	853	17.1	73.3	165.5	
Floor 35	927	18.5	73.3	165.5	
Floor 34	1001	20.0	73.3	165.5	
Floor 33	1076	21.5	73.3	165.5	
Floor 32	1151	23.0	73.3	165.5	
Floor 31	1227	24.5	73.3	165.5	
Floor 30	1303	26.1	73.3	165.5	
Floor 29	1379	27.6	73.3	165.5	
Floor 28	1455	29.1	73.3	165.5	
Floor 27	1535	30.7	73.3	165.5	
Floor 26	1615	32.3	73.3	165.5	
Floor 25	1695	33.9	73.3	165.5	
Floor 24	1768	35.4	75.4	178.4	
Floor 23	1874	37.5	161.0	272.6	
Floor 22	1936	38.7	73.3	165.5	
Floor 21	1998	40.0	73.3	165.5	
Floor 20	2051	41.0	73.3	143.2	
Floor 19	2103	42.1	73.3	143.2	
Floor 18	2153	43.1	73.3	143.2	
Floor 17	2202	44.0	73.3	143.2	
Floor 16	2256	45.1	75.0	169.8	
Floor 15	2317	46.3	75.6	178.4	
Floor 14	2359	47.2	69.0	143.2	
Floor 13	2401	48.0	69.0	143.2	
Floor 12	2443	48.9	69.0	143.2	
Floor 11	2485	49.7	69.0	143.2	
Floor 10	2532	50.6	69.0	143.2	
Floor 9	2579	51.6	69.0	143.2	
Floor 8	2626	52.5	69.0	143.2	
Floor 7	2605	52.1	NC	98.5	
Floor 6	2649	53.0	224.9	8.1	
Floor 5	4612	92.2	NC	NC	
Floor 4	4644	92.9	207.6	11.2	
Floor 3	5171	103.4	NC	11.2	
Floor 2	5211	104.2	321.5	8.1	

Table 4.19 Column 75 Lateral Bracing Code Check (orange indicates code violations)

Table 4.19	Column 75 Lateral	Bracing Code Check	κ (orange indicates co	ode violations)	
			Total Bracing Cap	acity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co	the Column	
Level	Load in Column	Force	North-South Direction	East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	58	1.2	6.0	138.9	
Floor 47	105	2.1	6.0	No info	
Floor 46	207	4.1	6.0	275.3	
Floor 45	279	5.6	6.0	156.1	
Floor 44	350	7.0	6.0	156.1	
Floor 43	351	-	-	-	
Floor 42	432	8.6	6.0	156.1	
Floor 41	433	-	-	-	
Floor 40	520	10.4	6.0	156.1	
Floor 39	593	11.9	6.0	156.1	
Floor 38	666	13.3	6.0	156.1	
Floor 37	739	14.8	6.0	156.1	
Floor 36	813	16.3	6.0	156.1	
Floor 35	888	17.8	6.0	156.1	
Floor 34	963	19.3	6.0	156.1	
Floor 33	1038	20.8	6.0	156.1	
Floor 32	1113	22.3	6.0	156.1	
Floor 31	1188	23.8	6.0	156.1	
Floor 30	1264	25.3	6.0	156.1	
Floor 29	1340	26.8	6.0	156.1	
Floor 28	1417	28.3	6.0	156.1	
Floor 27	1496	29.9	6.0	156.1	
Floor 26	1576	31.5	6.0	156.1	
Floor 25	1656	33.1	6.0	156.1	
Floor 24	1739	34.8	6.0	156.1	
Floor 23	1831	36.6	14.0	245.2	
Floor 22	1922	38.4	6.0	156.1	
Floor 21	2013	40.3	6.0	156.1	
Floor 20	2089	41.8	6.0	156.1	
Floor 19	2164	43.3	6.0	156.1	
Floor 18	2233	44.7	6.0	156.1	
Floor 17	2303	46.1	6.0	156.1	
Floor 16	2372	47.4	6.0	156.1	
Floor 15	2442	48.8	6.0	156.1	
Floor 14	2512	50.2	6.0	156.1	
Floor 13	2582	51.6	6.0	156.1	
Floor 12	2652	53.0	6.0	156.1	
Floor 11	2722	54.4	6.0	156.1	
Floor 10	2799	56.0	6.0	156.1	
Floor 9	2876	57.5	6.0	156.1	
Floor 8	2952	59.0	6.0	156.1	
Floor 7	2831	56.6	NC	261.4	
Floor 6	2952	59.0	121.1	NC	
Floor 5	2594	59.0	NC	NC NC	
Floor 4	2470	49.4	121.1	NC NC	
Floor 4	3363	67.3	NC	NC NC	
		ļ		NC NC	
Floor 2	3769	75.4	200.4	INC	

Table 4.20 Column 76 Lateral Bracing Code Check (orange indicates code violations)

Table 4.20	Column /6 Lateral	Bracing Code Check	ለ (orange indicates co		
			Total Bracing Cap	acity Provided	
Floor	Design Compression	Required 2% Bracing	to the Co		
Level	Load in Column	Force	North-South Direction	East-West Direction	
	[Kip]	[Kip]	[Kip]	[Kip]	
Roof	214	4.3	104.7	204.9	
Floor 47	305	6.1	75.2	175.5	
Floor 46	495	9.9	135.0	371.7	
Floor 45	633	12.7	6.0	116.2	
Floor 44	772	15.4	6.0	116.2	
Floor 43	774	-	-	-	
Floor 42	914	18.3	6.0	116.2	
Floor 41	916	-	-	-	
Floor 40	1057	21.1	6.0	116.2	
Floor 39	1197	23.9	6.0	116.2	
Floor 38	1338	26.8	6.0	116.2	
Floor 37	1479	29.6	6.0	116.4	
Floor 36	1620	32.4	6.0	116.4	
Floor 35	1762	35.2	6.0	116.2	
Floor 34	1904	38.1	6.0	116.2	
Floor 33	2046	40.9	6.0	116.2	
Floor 32	2189	43.8	6.0	116.2	
Floor 31	2331	46.6	6.0	116.2	
Floor 30	2475	49.5	6.0	116.2	
Floor 29	2618	52.4	6.0	116.2	
Floor 28	2762	55.2	6.0	116.2	
Floor 27	2906	58.1	6.0	116.2	
Floor 26	3051	61.0	6.0	116.2	
Floor 25	3196	63.9	6.0	116.2	
Floor 24	3342	66.8	71.1	53.2	
Floor 23	3523	70.5	92.1	53.2	
Floor 22	3704	74.1	92.1	53.2	
Floor 21	3886	77.7	92.1	53.2	
Floor 20	4033	80.7	92.1	53.2	
Floor 19	4179	83.6	92.1	53.2	
Floor 18	4330	86.6	92.1	53.2	
Floor 17	4481	89.6	92.1	53.2	
Floor 16	4632	92.6	92.1	53.2	
Floor 15	4784	95.7	92.1	59.0	
Floor 14	4938	98.8	92.1	59.0	
Floor 13	5091	101.8	92.1	59.0	
Floor 12	5246	104.9	92.1	59.0	
Floor 11	5400	108.0	92.1	59.0	
Floor 10	5551	111.0	92.1	59.0	
Floor 9	5707	114.1	92.1	59.0	
Floor 8	5865	117.3	92.1	59.0	
Floor 7	-	-	-	-	
Floor 6	_	_	-	_	
Floor 5	_	_	-	_	
Floor 4	_	_	-	_	
Floor 3	_	_	_	_	
Floor 2	_	_	-	_	

Table 4.21 Column 77 Lateral Bracing Code Check (no code violations)

Table 4.21	Column // Lateral	Bracing Code Check	(no code violations)	
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	156	3.1	69.0	165.5
Floor 47	235	4.7	73.3	165.5
Floor 46	314	6.3	69.0	165.5
Floor 45	390	7.8	69.0	165.5
Floor 44	467	9.3	69.0	165.5
Floor 43	468	-	-	-
Floor 42	546	10.9	69.0	165.5
Floor 41	548	-	-	-
Floor 40	626	12.5	69.0	165.5
Floor 39	703	14.1	69.0	165.5
Floor 38	781	15.6	69.0	165.5
Floor 37	858	17.2	69.0	165.5
Floor 36	937	18.7	69.0	165.5
Floor 35	1015	20.3	69.0	165.5
Floor 34	1094	21.9	69.0	165.5
Floor 33	1174	23.5	69.0	165.5
Floor 32	1254	25.1	69.0	165.5
Floor 31	1333	26.7	69.0	165.5
Floor 30	1414	28.3	69.0	165.5
Floor 29	1494	29.9	69.0	165.5
Floor 28	1575	31.5	69.0	165.5
Floor 27	1656	33.1	69.0	165.5
Floor 26	1737	34.7	69.0	165.5
Floor 25	1818	36.4	69.0	165.5
Floor 24	1900	38.0	69.0	178.4
Floor 23	2001	40.0	71.1	178.4
Floor 22	2103	42.1	71.1	165.5
Floor 21	2206	44.1	71.1	165.5
Floor 20	2288	45.8	71.1	165.5
Floor 19	2372	47.4	71.1	165.5
Floor 18	2459	49.2	71.1	165.5
Floor 17	2547	50.9	71.1	165.5
Floor 16	2634	52.7	71.1	165.5
Floor 15	2722	54.4	71.1	165.5
Floor 14	2810	56.2	71.1	165.5
Floor 13	2899	58.0	71.1	165.5
Floor 12	2987	59.7	71.1	165.5
Floor 11	3076	61.5	71.1	165.5
Floor 10	3162	63.2	71.1	165.5
Floor 9	3253	65.1	71.1	165.5
Floor 8	3343	66.9	71.1	165.5
Floor 7	-	-	-	-
Floor 6	-	-	-	-
Floor 5	-	-	-	-
Floor 4	-	-	-	-
Floor 3	-	-	-	-
Floor 2	-	-	-	-

Table 4.22 Column 78 Lateral Bracing Code Check (no code violations)

Table 4.22	Column /8 Lateral	Bracing Code Check	(no code violations)	
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	59	1.2	69.0	138.9
Floor 47	114	2.3	69.0	165.5
Floor 46	183	3.7	71.1	218.6
Floor 45	265	5.3	69.0	143.2
Floor 44	347	6.9	69.0	143.2
Floor 43	348	-	-	-
Floor 42	431	8.6	69.0	No info
Floor 41	433	-	-	-
Floor 40	516	10.3	69.0	No info
Floor 39	598	12.0	69.0	No info
Floor 38	681	13.6	69.0	No info
Floor 37	763	15.3	69.0	143.2
Floor 36	847	16.9	69.0	143.2
Floor 35	930	18.6	69.0	143.2
Floor 34	1015	20.3	69.0	143.2
Floor 33	1099	22.0	69.0	143.2
Floor 32	1184	23.7	69.0	143.2
Floor 31	1269	25.4	69.0	143.2
Floor 30	1355	27.1	69.0	143.2
Floor 29	1440	28.8	69.0	143.2
Floor 28	1526	30.5	69.0	143.2
Floor 27	1612	32.2	69.0	143.2
Floor 26	1699	34.0	69.0	143.2
Floor 25	1785	35.7	69.0	143.2
Floor 24	1874	37.5	69.0	178.4
Floor 23	1980	39.6	69.0	178.4
Floor 22	2086	41.7	69.0	178.4
Floor 21	2193	43.9	69.0	178.4
Floor 20	2281	45.6	69.0	178.4
Floor 19	2370	47.4	69.0	178.4
Floor 18	2457	49.1	69.0	178.4
Floor 17	2545	50.9	69.0	178.4
Floor 16	2632	52.6	69.0	178.4
Floor 15	2720	54.4	69.0	178.4
Floor 15 Floor 14	2808	56.2	69.0	178.4
Floor 14 Floor 13	2808	56.2	69.0	178.4
Floor 13 Floor 12	2985	57.9 59.7	69.0	
Floor 12 Floor 11				178.4 178.4
Floor 11 Floor 10	3073	61.5	69.0	+
	3164	63.3	69.0	178.4
Floor 9	3254	65.1	69.0	178.4
Floor 8	3345	66.9	69.0	178.4
Floor 7	-	-	-	-
Floor 6	-	-	-	-
Floor 5	-	-	-	-
Floor 4	-	-	-	-
Floor 3	-	-	-	-
Floor 2	-	-	-	-

Table 4.23 Column 79 Lateral Bracing Code Check (orange indicates code violations)

1able 4.23	Column 79 Lateral	Bracing Code Check	(orange indicates co	ide violations)
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	342	6.8	159.9	33.3
Floor 47	524	10.5	52.3	29.6
Floor 46	703	14.1	52.3	11.7
Floor 45	885	17.7	52.3	11.7
Floor 44	1084	21.7	52.3	11.7
Floor 43	1087	-	-	-
Floor 42	1288	25.8	52.3	11.7
Floor 41	1291	-	-	-
Floor 40	1476	29.5	52.3	11.7
Floor 39	1659	33.2	52.3	11.7
Floor 38	1844	36.9	52.3	11.7
Floor 37	2028	40.6	52.3	11.7
Floor 36	2213	44.3	52.3	11.7
Floor 35	2398	48.0	52.3	11.7
Floor 34	2584	51.7	52.3	11.7
Floor 33	2769	55.4	52.3	11.7
Floor 32	2956	59.1	52.3	11.7
Floor 31	3142	62.8	52.3	11.7
Floor 30	3329	66.6	52.3	11.7
Floor 29	3516	70.3	52.3	11.7
Floor 28	3704	74.1	52.3	11.7
Floor 27	3891	77.8	52.3	11.7
Floor 26	4080	81.6	52.3	11.7
Floor 25	4269	85.4	52.3	11.7
Floor 24	4459	89.2	52.3	11.7
Floor 23	4696	93.9	52.3	11.7
Floor 22	4935	98.7	52.3	11.7
Floor 21	5175	103.5	52.3	11.7
Floor 20	5366	107.3	52.3	11.7
Floor 19	5558	111.2	52.3	11.7
Floor 18	5750	115.0	52.3	11.7
Floor 17	5943	118.9	52.3	11.7
Floor 16	6138	122.8	52.3	11.7
Floor 15	6333	126.7	52.3	11.7
Floor 14	6528	130.6	52.3	11.7
Floor 13	6723	134.5	52.3	11.7
Floor 12	6918	138.4	52.3	11.7
Floor 11	7114	142.3	52.3	11.7
Floor 10	7310	146.2	52.3	11.7
Floor 9	7508	150.2	52.3	11.7
Floor 8	7706	154.1	52.3	11.7
Floor 7	7962	159.2	193.8	8.1
Floor 6	8103	162.1	117.4	6.9
Floor 5	8500	170.0	165.9	12.9
Floor 4	8797	175.9	303.2	8.6
Floor 3	9027	180.5	259.7	8.5
Floor 2	9220	184.4	No info	-

Table 4.24 Column 80 Lateral Bracing Code Check (orange indicates code violations)

Table 4.24	Column 80 Lateral	Bracing Code Check	ለ (orange indicates co	ode violations)
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	314	6.3	115.3	18.5
Floor 47	441	8.8	115.3	10.6
Floor 46	569	11.4	115.3	10.6
Floor 45	696	13.9	115.3	10.6
Floor 44	831	16.6	115.3	10.6
Floor 43	833	-	-	-
Floor 42	970	19.4	115.3	10.6
Floor 41	973	-	-	-
Floor 40	1102	22.0	115.3	10.6
Floor 39	1230	24.6	115.3	10.6
Floor 38	1359	27.2	115.3	10.6
Floor 37	1488	29.8	115.3	10.6
Floor 36	1618	32.4	115.3	10.6
Floor 35	1748	35.0	115.3	10.6
Floor 34	1878	37.6	115.3	10.6
Floor 33	2008	40.2	115.3	10.6
Floor 32	2139	42.8	115.3	10.6
Floor 31	2271	45.4	115.3	10.6
Floor 30	2402	48.0	115.3	10.6
Floor 29	2534	50.7	115.3	10.6
Floor 28	2666	53.3	115.3	10.6
Floor 27	2798	56.0	115.3	10.6
Floor 26	2931	58.6	115.3	10.6
Floor 25	3064	61.3	115.3	10.6
Floor 24	3198	64.0	115.3	10.6
Floor 23	3363	67.3	115.3	10.6
Floor 22	3529	70.6	115.3	10.6
Floor 21	3695	73.9	115.3	10.6
Floor 20	3828	76.6	115.3	10.6
Floor 19	3960	79.2	115.3	10.6
Floor 18	4096	81.9	115.3	10.6
Floor 17	4231	84.6	115.3	10.6
Floor 16	4368	87.4	115.3	10.6
Floor 15	4505	90.1	115.3	10.6
Floor 14	4642	92.8	115.3	10.6
Floor 13	4780	95.6	115.3	10.6
Floor 12	4918	98.4	115.3	10.6
Floor 11	5057	101.1	115.3	10.6
Floor 10	5194	103.9	115.3	10.6
Floor 9	5334	106.7	115.3	42.0
Floor 8	5475	109.5	115.3	42.0
Floor 7	5775	115.5	180.6	12.1
Floor 6	5798	116.0	10.3	3.2
Floor 5	7964	159.3	274.2	NC
Floor 4	8168	163.4	NC	237.8
Floor 3	8368	167.4	187.7	6.4
Floor 2	8498	170.0	No info	-

Table 4.25 Column 81 Lateral Bracing Code Check (orange indicates code violations)

Table 4.25	Column 81 Lateral	Bracing Code Check	: (orange indicates co	de violations)
			Total Bracing Cap	acity Provided
Floor	Design Compression	Required 2% Bracing	to the Co	lumn
Level	Load in Column	Force	North-South Direction	East-West Direction
	[Kip]	[Kip]	[Kip]	[Kip]
Roof	87	1.7	10.3	146.3
Floor 47	176	3.5	8.1	168.6
Floor 46	277	5.5	8.1	303.3
Floor 45	405	8.1	50.1	143.2
Floor 44	536	10.7	50.1	143.2
Floor 43	539	-	-	-
Floor 42	671	13.4	50.1	No info
Floor 41	673	-	-	-
Floor 40	802	16.0	50.1	No info
Floor 39	930	18.6	50.1	No info
Floor 38	1058	21.2	50.1	No info
Floor 37	1187	23.7	52.3	168.6
Floor 36	1316	26.3	52.3	168.6
Floor 35	1446	28.9	50.1	168.6
Floor 34	1576	31.5	50.1	168.6
Floor 33	1706	34.1	50.1	168.6
Floor 32	1837	36.7	50.1	168.6
Floor 31	1968	39.4	52.3	168.6
Floor 30	2099	42.0	52.3	168.6
Floor 29	2231	44.6	52.3	168.6
Floor 28	2363	47.3	52.3	168.6
Floor 27	2495	49.9	50.1	168.6
Floor 26	2628	52.6	50.1	168.6
Floor 25	2761	55.2	50.1	168.6
Floor 24	2895	57.9	50.1	168.6
Floor 23	3061	61.2	52.3	168.6
Floor 22	3228	64.6	52.3	168.6
Floor 21	3395	67.9	52.3	143.2
Floor 20	3530	70.6	52.3	143.2
Floor 19	3665	73.3	52.3	143.2
Floor 18	3799	76.0	52.3	143.2
Floor 17	3934	78.7	52.3	143.2
Floor 16	4069	81.4	52.3	143.2
Floor 15	4205	84.1	50.1	143.2
Floor 14	4341	86.8	50.1	143.2
Floor 13	4477	89.5	50.1	143.2
Floor 12	4613	92.3	50.1	143.2
Floor 11	4750	95.0	50.1	94.3
Floor 10	4888	97.8	50.1	94.3
Floor 9	5026	100.5	50.1	94.3
Floor 8	5164	100.5	50.1	94.3
Floor 7	5463			10.0
Floor 6		109.3 109.5	270.7 173.0	115.0
Floor 5	5475 6041	109.5		166.9
Floor 5 Floor 4	<u> </u>		262.4	166.9
	6056	-		
Floor 3	6270	125.4	103.1	21.2
Floor 2	6351	127.0	No info	32.5

### 4.3 Summary of Structural Vulnerabilities

The following statements summarize the conclusions reached in the assessment of the structural vulnerabilities and lateral code bracing violations of WTC7:

- The WTC7 structure was required to comply with the 2% lateral bracing provision in Section C26-1001.2 of the Building Code of the City of New York.
- The girders framing into the interior columns were responsible for providing lateral bracing to the interior columns because the concrete slab was neither designed to provide nor capable of providing bearing resistance to the column.
- The axial capacity of the girders in tension and compression was dependent upon their connection details.
- The Engineer of Record did not provided the contractor's fabricator with the necessary axial design forces to comply with the 2% code requirement.
- The welded double-angle knife connections selected by the design team to connect over half the girders and beams to interior columns were weak in tension and not adequate as lateral bracing. Simple hand calculations using AISC allowable design procedures would have demonstrated that welded double-angle knife connections were not capable of being designed for the tensile loads required to brace from one side many of the heavily loaded columns in the WTC7 building including Column 79. Therefore these connections were an inappropriate choice to use as the lateral bracing for these columns.
- The combination of welded double-angle girder-to-column connections, which were weak in tension, with three-sided girder bracing at many interior columns resulted in numerous locations where the columns were insufficiently laterally supported.
- Approximately 46% of all floor-to-interior column joints in the building did not meet the 2% lateral bracing code requirement in at least one direction.
   Furthermore, 75% of the interior columns possessed at least one lateral bracing code violation.
- In addition to the widespread lateral bracing code violations, other characteristics made the building less robust and redundant and particularly vulnerable to disproportionate collapse including the use of multiple interdependent transfer structures, trench headers, and large tributary floor areas.
- The code violations and the other identified structural vulnerabilities caused the progression of global collapse on 11 September 2001 as explained in Section 5.0.

# 5.0 PROBABLE GLOBAL COLLAPSE MECHANISM

Although the precise details of the WTC7 collapse cannot be accurately simulated by a computer analysis, probable stages of the collapse can be identified using a combination of information gained from parametric structural computations and an analysis of the visual evidence of the collapse.

While Section 4.0 evaluates the inherent vulnerability of the WTC7 structure, this section describes the way in which the building failed as a result of these vulnerabilities. The global collapse mechanism, which was initiated by the local failure of a single girder and resulted in the total collapse of the building, is described in six stages. By nature of the collapse event, the first several stages of the sequence are considered with a much higher level of analysis detail than the later stages. The final stages are considered more broadly because the behavior of the structure during these stages is highly dynamic and chaotic with many complex events occurring simultaneously.

This section of the report also demonstrates that a disproportionate building collapse would not have occurred on 11 September 2001 as a result of the local failure had these vulnerabilities not existed.

#### 5.1 Summary of Visual Evidence of Collapse

Video footage exists of the WTC7's collapse; however, most of the footage shows only the north façade of the building, and all of it was taken at or near street level at a distance from the site. As a result, there is no available visual evidence of the behavior of the south facade of the building, the lower third of the building (due to obstructing surrounding buildings), or the roof of the building from a birds-eye view during the collapse. From the available footage of the north façade, however, several key indicators of the behavior of the building during the collapse are evident.

The first indicator is the fall of the East Penthouse structure. A vertical kink, which aligns approximately with Columns 79 and 80, forms in the penthouse. Subsequently, the two sides of the East Penthouse structure tip inwards towards the kink and disappear into the building below (Figure 5.1A). An approximate 5 second pause follows this event, during which time no significant activity is observed although light is evident through the east side of the building indicating that it has been partially hollowed-out (Figure 5.1B). The pause is then succeeded by a rapid progression of collapse of the remaining penthouse structures to the west (Figure 5.1C), followed immediately by the onset of global collapse of the perimeter structure. The collapse of the interior structure west of Column 76 precedes the collapse of the building's perimeter by approximately one story (Figure 5.1D). As the entire building falls, a horizontal "kink" is observed in the north façade of the building which is aligned approximately with Column 48 on the

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perimeter (Figure 5.2). It is likely that this kink is an indication of the northward movement of the upper floors of the eastern region of the building. These visual indicators aid in the reconstruction of the probable global collapse sequence because they relate the results of the studies to tangible facts.



Figure 5.1 Views of North façade of WTC7 during collapse from available video footage (A) fall of East Penthouse (B) approx 5 second pause following East Penthouse fall (C) start of westward collapse of remainder of the interior of the building followed immediately by (D) collapse of entire structure



Figure 5.2 Still from available video footage showing horizontal "kink" in north face of building during collapse

## 5.2 Basis of Staged Deconstruction

Each stage in the global collapse sequence represents an approximate "snapshot" of the structure during the collapse. These snapshots are intended to identify moments in time when the structure was particularly susceptible to failure given its inherent vulnerabilities. Although the global collapse analysis has been separated into distinct events, visual evidence shows that some of the events occurred either simultaneously or in rapid succession. The sequence of these stages and the grouping of events in each stage is an effort to capture the likely progression of collapse and to illustrate causal effects of earlier stages of collapse. It should not be misinterpreted as a representation of the exact sequence of events or an implication of the timing of the events.

## 5.3 Summary of Probable Global Collapse Mechanism

Based on both the results of the structural studies and observation of the visual evidence of the collapse, a probable global collapse sequence was established. The sequence, illustrated in Figures 5.3 to 5.8, may be summarized in six stages. Because two possible initiating local collapse events were identified in the report by Dr Colin Bailey (Ref 7), the probable global collapse sequence includes two versions of Stages 1 and 2 (labeled Scenario A or Scenario B) in Table 5.1:

Table 5.1 Summary of probable global collapse sequence

STAGE	INTERIOR EVENT	EXTERIOR EVENT
Initiating Failure	Based on the results of the structural fire studies documented in the Bailey report, the triggering event is either the unseating of Girder 44-79 at its connection to Column 79 at Floor 13 (Scenario A) or at Floor 10 (Scenario B). A failure on Floor 13 corresponds to a failure during the cooling phase of the fire and a failure on Floor 10 corresponds to a failure during the heating phase. In both cases it is found that the two other connections to Column 79 remain intact immediately following the unseating of the girder.	No event
1A (Fig 5.3)	Scenario A: On Floor 13, the floor framing and floor slabs supported by Girder 44-79 break off (in some cases along trench headers) and fall, impacting the floor below and leading to its failure and the progressive failure of the northeast slabs on all lower floor levels to the ground. The corner slabs on these levels likely remain in place but with considerable damage.	No event
1B (similar to Fig 5.3)	Scenario B: On Floor 10, the floor framing and floor slabs supported by Girder 44-79 break off (in some cases along trench headers) and fall, impacting the floor below and leading to its failure and the progressive failure of the northeast slabs on all lower floor levels to the ground. The corner slabs on these levels likely remain in place but with considerable damage.	No event

Table 5.1 cont Summary of probable global collapse sequence

STAGE	INTERIOR EVENT	EXTERIOR EVENT
2A (Fig 5.4)	Scenario A: The floor collapse leaves a void to the north and east of Column 79. As a result, the western knife connections between the column and Girder 76-79, which provide insufficient lateral bracing to Column 79, fracture and trigger the buckling of the column below Floor 14. All floor areas tributary to Column 79 begin to collapse.	
2B (similar to Fig 5.4)	Scenario B: For the same reasons described in Stage 2A above, the floor collapse from Floor 10 to the ground triggers the buckling of Column 79 below Floor 11. All floor areas tributary to Column 79 begin to collapse.	No event
3 (Fig 5.5)	The onset of buckling at Column 79 and the failure of its tributary floor areas creates a void to the north and east of Column 80 and causes it to buckle approximately between Floors 7-20 as its inadequate western knife connections fracture. The floor areas supported by both Columns 79 and 80 (including the East Penthouse) break off at trench headers and fall to the ground as these columns lose the ability to carry load. It takes approximately 5 seconds for the uppermost floor areas supported by Columns 79 and 80 to reach the elevation of Transfer Trusses 1 and 2 at Floor 7. During this period, the eastern portions of these trusses may have been impacted by portions of the falling floor slabs, causing increasingly significant damage to the trusses.	No event

Table 5.1 cc	ont Summar	v of pro	obable o	ilobal co	ollanse sed	luence
Tuoic o. i cc	Jill Jallillal	y OI PI	Journe d	noour co	Jiiupse see	Jucince

	LINITEDIOD EVENIT	
STAGE	INTERIOR EVENT	EXTERIOR EVENT
4	The eastern-most diagonals and	Following failure of the eastern
(Fig 5.6)	supports of Transfer Trusses 1 and 2	diagonal of Transfer Truss 1, tension
	are the most exposed to impact	force increases considerably in
	from the falling floor slabs and	Girder 73–76 and its connection to
	sustain sufficient damage to fail,	Column 73 at Floor 7 fails. The
	which results in the total failure of	failure results in loss of back-span
	the trusses (see Exterior Events for	support for two cantilevered
	more detail). Columns 76, 77 and	transfer girders framing into it
	78, which are supported on the	which support perimeter Columns
	failed transfer trusses, and their	47 and 48. As a consequence, the
	floor slabs begin to collapse.	transfer girders rotate, shedding
		load to perimeter Columns 46 and
		49 and deforming the perimeter belt
		trusses. Column 46 and its adjacent
		perimeter framing including
		Columns 44 and 45 buckle over the
		lower floors due to the increased
		load and the loss of lateral support
		from the interior floors. The
		northern columns on the eastern
		perimeter then begin to buckle in a
		similar manner.
5	The falling floor areas tributary to	As the northeast corner of the
(Fig 5.7)	Columns 76-78 impose an eastern	perimeter frame buckles over its
	horizontal force on the remaining	lower floors, the upper region of the
	intact floors to the west. The intact	frame sways northward creating a
	floors are susceptible to rupture due	horizontal "kink" visible on the north
	to their pre-segmentation by the	façade of the building. The
	trench headers and core openings,	perimeter frame buckling at the
	and the horizontal forces cause	base of the building spreads to the
	them to break apart in the	south and west as the weight of the
	horizontal plane at the boundaries	perimeter walls shifts to adjacent
1	of the trench headers. The resulting	stable supports, overloads them and
	lateral displacements of the slab	causes them to fail in rapid
	segments cause Columns 64-75 to	succession.
	lose stability above Floor 7.	3.555556111
	Simultaneously Column 81 buckles	
	due to fracture of its east	
	connections following the loss of	
	the floor areas tributary to Column	
	78.	
1	/0.	

Table 5.1 cont. Summary of probable global collapse sequence

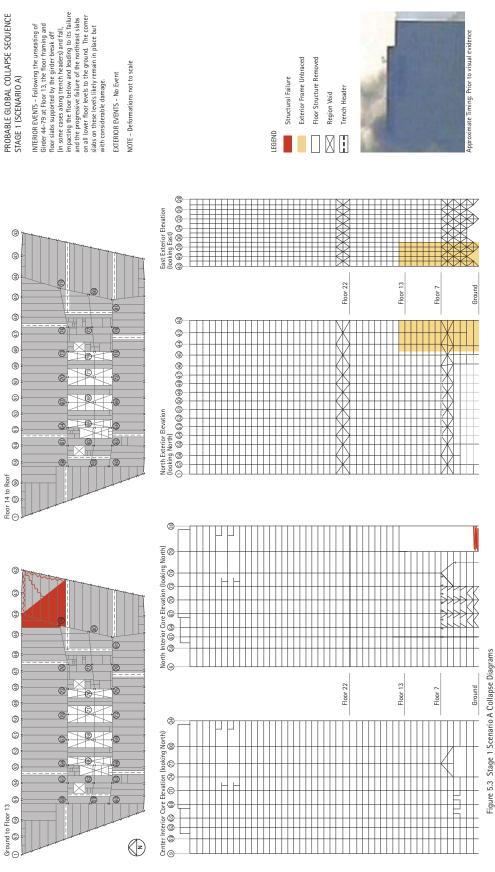
14016 5.1 601	it summary of probable global conaps	e sequence
STAGE	INTERIOR EVENT	EXTERIOR EVENT
6	As Columns 64 through 75 and their	The rapid western spread of
(Fig 5.8)	tributary floor areas fall, the transfer	perimeter buckling at the base of
	girders at Floor 7 supporting the	the building continues on both its
	north façade fail.	north and south sides. The loss of
		the transfer girders exacerbates the
		failures on the north perimeter.
		Ultimately the spread of perimeter
		buckling reaches the western side of
		the building and fails the remaining
		structure.

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**Guy Nordenson and Associates** 



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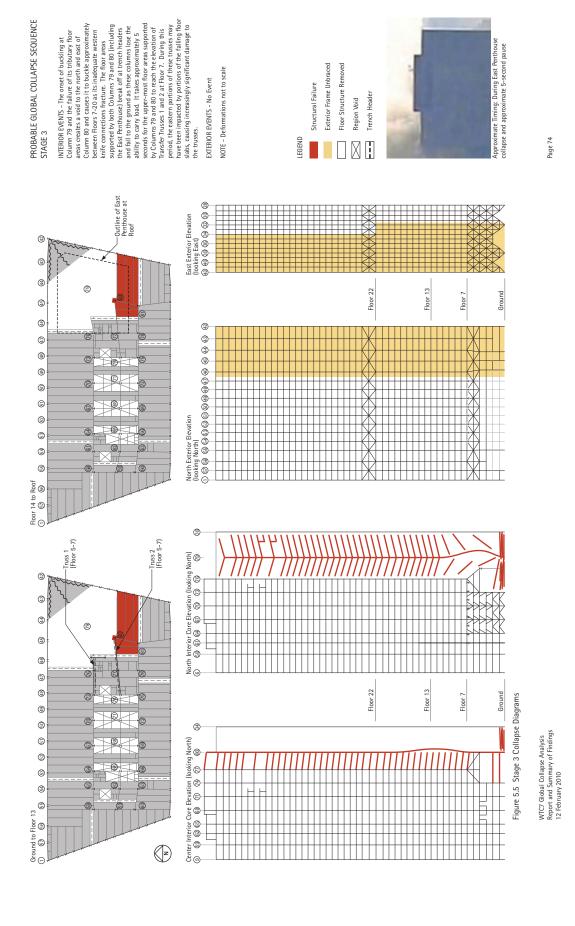


**Guy Nordenson and Associates** 

## INTERIOR EVENTS – The floor collapse leaves a void to the north and east of Column 39. As a result, the western white connections between the column and Girder 76–79, which provide insufficient lateral bracing to Column 79, fracture and trigger the buckling of the column below Floor 14. All floor areas tributary to Column 79 begin to collapse. PROBABLE GLOBAL COLLAPSE SEQUENCE STAGE 2 (SCENARIO A) Floor Structure Removed Exterior Frame Unbraced NOTE - Deformations not to scale EXTERIOR EVENTS - No Event Structural Failure Trench Header Region Void -Outline of East Penthouse at Roof East Exterior Elevation (looking East) 주의 수의 수의 수의 수의 수의 (<del>)</del> 7 @ (3) Floor 22 Floor 13 Floor 7 Ground ; (3) (4) (9) (3) (#) 0 (4) (3) **(** (3) (3) (3) (3) Floor 14 to Roof (1) (5) (6) 8 North Interior Core Elevation (looking North) © © © © © © © © © © @ 7 9 3 (4) (9) Figure 5.4 Stage 2 Scenario A Collapse Diagrams (3) (3) Floor 22 Floor 13 Ground 9 6 (3) 6 Center Interior Core Elevation (looking North) 9 6 6 6 6 70 49 70 60 (3) **(B)** 0, 3 (3) Ground to Floor 13 $\mathbb{Z}$

JA-3976









(2)

(4)

(<del>2</del>) (9)

**(**<del>4</del>**) a** 

(2)

(3) (E) (23) (23) (2) (93)

Upper Floors

-Truss 1 (Floor 5-7)

**4** (4)

(9) (4) (4)

**Q** 

(20)

(2)

(23) (23) (Z) (3)

Lower Floors

INTERIOR EVENTS – The eastern–most diagonals and supports of Transfer Trusses I and 2 are the most exposed to impact from the falling floor slabs and sustain sufficient damage to fail, which results in the total failure of the trusses (see Exterior Events for more detail). Columns 76, 77 and 78, which are supported on the failed transfer trusses, and their floor slabs begin to collapse.

As a consequence, the transfer girdes notate, steeding load to permeter Columns 46 and 49 and deforming the perimeter belt trusses. Column 46 and its adjacent perimeter framing including Columns 44 and 45 buckle over the lover floors due to the increased load and the loss of lateral due to the increased load and the loss of lateral increases considerably in Girder 73-76 and its connection to Column 73 at Floor 7 fails. The failure results in loss of back-span support for two cartilevered transfer girders framing into it which support perimeter Columns 47 and 48. EXTERIOR EVENTS - Following failure of the eastern diagonal of Transfer Truss 1, tension force columns on the eastern perimeter then begin to support from the interior floors. The northern buckle in a similar manner.

East Exterior Elevation (looking East)

(<del>4</del>)

North Exterior Elevation (looking North) 订蚜矈 医唇唇容弱卵骨锤砂锤

in Interior Core Elevation (looking North)

(3)

Center Interior Core Elevation (looking North)

 $\mathbb{Z}$ 

Truss 2 (Floor 5-7)

NOTE - Deformations not to scale and buckled shape of perimeter frame not shown for clarity.

LEGEND

Exterior Frame Unbraced Floor Structure Removed Structural Failure

Region Void

Trench Header

Floor 22

Floor 22

Floor 13

Floor 7

Floor 13

Floor 7

Approximate Timing: At onset of collapse of remaining interior structure

Section A (Stage 4C) R.0087 Ground

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Stage 4C

















































































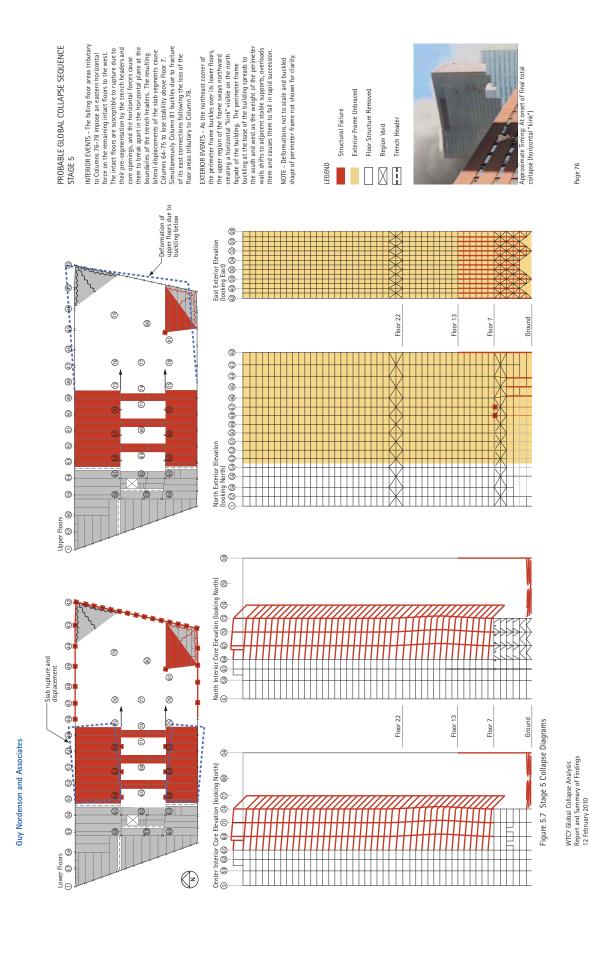




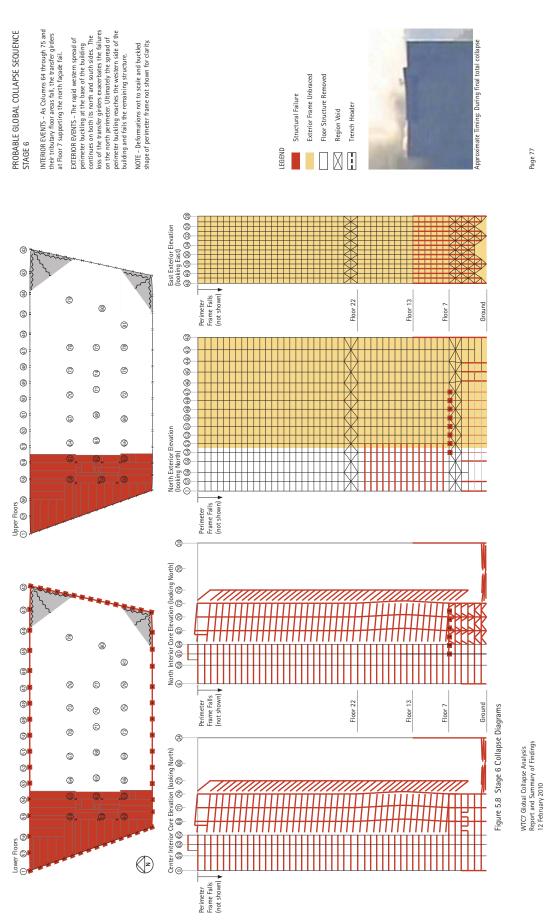




Ground







JA-3980

#### 5.4 Probable Collapse Sequence Stage 1 Analysis Details

Stage 1 consists of the progressive collapse of the floor structure in the northeast corner of the building from either Floor 13 or Floor 10 to the ground following the unseating of Girder 44–79 to Column 79 at either Floor 13 (Scenario A, Figure 5.3) or 10 (Scenario B).

Using the calculation methodology outlined in Section 5.4.1 and explained in detail in Appendix B, it was determined that the unseating of Girder 44–79 to Column 79 at Floor 13 results in an impact force on Floor 12 large enough to fail its seated connection to Column 79 in shear. The failed girder on Floor 12 then pulls down its tributary floor structure, impacting and failing the floor below it in a similar manner. This behavior propagates to the ground because no floor is capable of arresting it, including Floors 5 and 7, which are thicker and more heavily reinforced than the other floors. The floor failures are aided by the presence of trench headers in the slabs which allow them to break off with limited energy dissipation or transfer of load to adjacent structure.

A detailed floor slab collapse analysis was carried out for Scenario A (ie an initiating event on Floor 13) only. Although no analysis was performed for Scenario B (ie an initiating event on Floor 10), the similarities between Floors 13 and 10 and the conservative decision to disregard energy accumulation in the analysis allow the results and conclusions of the analysis to apply to Scenario B.

The analysis determined the failure of Girder 44–79 at Floor 13, or at a lower floor in the building, led to localized floor collapse on all subsequent lower levels to the ground, which constitutes unacceptable performance from a disproportionate collapse prevention standpoint.

#### 5.4.1 Scenario A Floor Collapse Analysis

The full details and calculations of the Floor Collapse Analysis are contained in Appendix B. This section provides an overview of the methodology and conclusions of the analysis for Scenario A, a local girder failure at Floor 13.

Upon failure of Girder 44–79's connection to Column 79 on Floor 13, the southern end of the girder would unseat, rotate and fall toward Floor 12. Using principles of energy conservation, it was determined that the energy of impact of the falling floor slab on Floor 12 would be sufficient to fail this floor and would cause the propagation of floor collapse to the floor below. Using the same methodology, it was determined that it would not be possible to arrest the propagation of floor collapse on subsequent lower levels, including Floors 5 and 7 which were thicker and more highly reinforced.

The basis for the analysis was an energy comparison between the remaining potential energy of a floor slab once it has deformed and broken away from its surrounding slab versus the energy required to fail the support structure of the floor below as follows:

A conservative approach to the analysis was taken in which the energy comparison was made on a relative floor-to-floor basis without allowing the potential energy of falling floors to accumulate. For example, once it was determined that Floor 12 would fail as a result of the impact of Floor 13, both the remaining potential energy of Floor 13 and its new potential energy due to its mass falling from Floor 12 to 11 were set to zero. Only the new potential energy of Floor 12 falling to Floor 11 was included in the next energy comparison at Floor 11 to assess whether the collapse would propagate further (Figure 5.9).

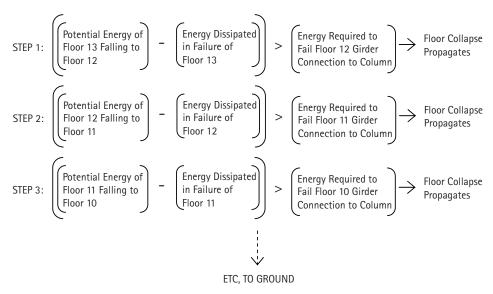


Figure 5.9 Conservative methodology for floor collapse assessment

While the study requires assumptions about geometry and deformation characteristics of the failing floor system, a conservative approach was taken to establish a lower-bound potential energy and an upper-bound deformation energy, thereby producing the lowest possible shear force transferred to the girder-to-column connection at each level.

As the basis for determining both the potential energy of the falling floor slab and the amount of energy dissipated in its failure, a structural analysis model in SAP2000 was generated to assess the probable geometry of each floor as it collapsed due to the failure of Girder 44–79 at Column 79. As the girder was unseated, it would have pulled down the floor beams and floor slab it supported. The roughly square floor slab area would have attempted to distort into a hyperbolic paraboloid-like shape as it fell because it remained supported on two sides while losing support on its other two sides due to fracture of the floor slab (Figure 5.10).

The potential energy of each falling floor was calculated as the mass of the floor tributary to the impact point under the deformed geometry condition multiplied by the height over which that mass would fall before impacting the floor below (Figure 5.11).

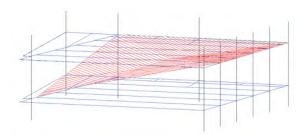


Figure 5.10 Deformed geometry of floor during collapse

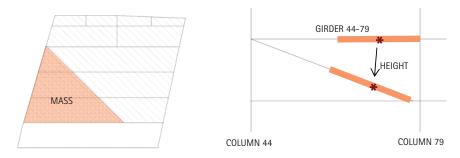


Figure 5.11 Basis for potential energy calculation at each floor level

The energy dissipated when the floor fell was the energy required to fracture the slab's continuity with the adjacent slab and to inelastically hinge the slab along yield lines to allow it to deform. These energies were calculated as either the fracture energy associated with rupture of the concrete and steel in the floor slab or the plastic energy

from moment-rotation curves for the floor slab. The sources of energy dissipation are as follows (illustrated in Figure 5.12):

- Tensile fracture of highly deformed bays directly to the south and west of the falling floor slab area
- Shear and tensile fracture of the floor sections framing into the south and west sides of Column 79
- Plastic hinging of the perimeter of the falling floor slab
- Plastic hinging of the borders with the south and west deformed bays (only where full slab depth trench headers are not present)
- Energy dissipation based on the rotational deformation of the falling floor slab area along slab hinge lines
- Plastic deformation of falling girder end at impact with floor below

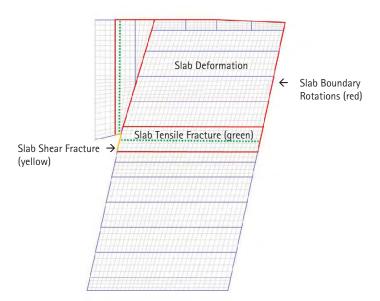


Figure 5.12 Assumed energy dissipation modes and locations (slab boundary rotations on southern boundary of floor failure only included where full-depth trench headers not present)

The deformed shape of the floor and the energy calculations account for the presence of a trench header on the southern boundary of the failed floor region. The presegmentation caused by the trench header reduced the energy dissipated by the detachment of the slab along this border.

Subtracting the total dissipated energy from the initial potential energy of the partial floor prior to collapse provided the potential energy of the floor at the moment of impact with Floor 12. Additional energy was dissipated by the inelastic deformation of the girder as its point of impact with Floor 12. The remaining potential energy was then converted to a static force based on the stiffness of the impact location and the resulting girder deflection. The resulting shear force transferred to the connection at Column 79 was then calculated and compared with the expected shear capacity of the connection to determine whether the connection would fail and cause Floor 12 to collapse. This series of calculations was performed for each floor level between Floor 12 and the ground.

The results of the analysis are summarized in the tables in Appendix B. It is evident from the results that for each floor level the impact of only the mass of the floor directly above it is sufficient to fail it and propagate the collapse, even at Floors 5 and 7. In reality, the impact force would be even larger due to the accumulated mass of higher floor levels, making this assessment highly conservative.

#### 5.5 Probable Collapse Sequence Stage 2 Analysis Details

In Stage 2 Column 79 buckles following the loss of its adjacent floor structure to the north and east including Girder 44–79, which framed into the northern flange of the column (Figure 5.4). In Scenario A, the column loses floor framing between Floor 13 and the ground. In Scenario B, floor structure is lost between Floor 10 and the ground. As a result of the floor loss, Column 79 becomes reliant on the girders framing into it from the south and west for lateral bracing over the height of the floor collapse.

Using the methodology outlined in Section 5.5.1 and Appendix C, it was determined that the loss of floor structure to the north and east of Column 79, either between Floor 13 and the ground or between Floor 10 and the ground, was sufficient to cause Column 79 to lose stability and buckle. In a well-designed building, a floor collapse adjacent to a column would not normally cause the column to buckle; however, the interior columns in WTC7 were vulnerable to lateral instability as a result of their improperly designed lateral bracing (see Section 4.2). Section 5.5.2 demonstrates that had Column 79 been provided with the code-required lateral bracing, it would not have buckled following the floor collapse.

As Column 79 buckles, it loses its ability to support vertical load, and the floor slabs and floor framing supported by it begin to fall, including the East Penthouse at the top of the building, as evidenced by the video footage of the collapse. On typical floor levels, the floor slabs break off to the west along two north-south running trench headers. The segmentation of the slab created by the trench headers likely allows the falling floor slabs to detach from the intact structure to the west with minimal transfer of load and

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damage. On the southern boundary of the falling floor slabs, no trench headers are present, and the floor slab fails along an east-west line at Column 80, resulting in loss of floor framing to the north of Column 80 and Stage 3 of the collapse (see Section 5.6).

#### 5.5.1 Column 79 Stability Analysis

The full details and calculations of the Interior Column Stability Analyses are contained in Appendix C. This section provides an overview of the analysis methodology that resulted in the conclusions drawn above.

All columns, including interior gravity columns such as Column 79, require lateral bracing at each floor level because they possess initial out-of-straightness due to the allowable fabrication and erection tolerances in the assembly of their components as well as eccentricities in loading. This crookedness imposes lateral forces on the floor levels when the columns are loaded axially (Figure 5.13). When a column has adequately stiff and strong lateral bracing, the secondary effects of the crookedness are negligible. However, if a column is not sufficiently braced, the effect of the crookedness may be amplified, leading to buckling (Ref 15).

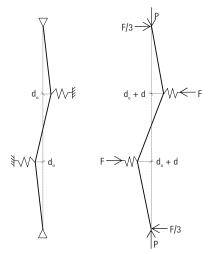


Figure 5.13 Example of lateral forces resulting from an initial crookedness of a column (Ref 15)

Following the collapse of the northeast floors including the loss of the girders framing into the north flange of Column 79, the column becomes dependent on the remaining girders framing into it from the south and west for lateral bracing (Figure 5.14). This bracing configuration imposes tensile forces on the welded double-angle knife

connections which connect these two girders at most levels to the column. As explained in Section 4.2.3, the concrete floor diaphragm to the south and west is not capable of providing lateral support to the column.

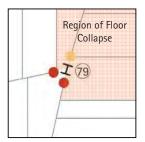


Figure 5.14 Remaining bracing configuration for Column 79 following floor failure (red = welded double-angle knife connections which are weak in tension)

The column stability analysis described in this section and in Appendix C is a means of assessing, using the methodology described in *Steel Structures* by William McGuire (Ref 15), whether the remaining lateral support to Column 79 was sufficiently strong to brace it and to allow it to continue to support gravity loads as the adjacent floor framing was lost.

The stability studies for Column 79, nonlinear analyses performed in SAP2000, considered a single full-height column with the material and sectional properties of the as-built column (a W14x730 built up with side plates). In order to perform a stability analysis, an initial out-of-straightness must be applied to the column. AISC design column bracing specifications use a slope of 1:500 to establish minimum brace forces. The 1:500 is consistent with the alignment tolerances for members with field splices in the Code of Standard Practice for Steel Buildings and Bridges (see Figure C-7.7, Ref 3 and Ref 2).

In reality, the out-of-straightness of Column 79 may have been greater than its initial erection out-of-straightness as a result of building movements and deformations of the column induced by the northeast floor failure described in Stage 1 of the collapse sequence and the thermal effects on the floor framing of fire on numerous levels of the building. Therefore, the crookedness of 1:500 used in the stability studies, without consideration for additional possible deformations due to the fires and floor failures, is considered to be a reasonable, if not conservative, estimate of the likely crookedness on the actual column immediately before buckling.

Because the configuration of the column's crookedness immediately following the initiation of sequential floor collapse in Stage 1 cannot be known, the stability analyses considered all possible crookedness configurations within the established 1:500 slope limit. The demonstration that any one of these configurations caused instability of a column was sufficient to establish that the column buckled. Therefore, to reduce the number of analysis iterations, the most critical out-of-straightness within the established 1:500 erection tolerance limit was identified and only this case was run.

For Scenario A, the lateral bracing of the column from Floor 14 to the roof (ie where the floors were assumed to be intact) was conservatively assumed to have infinite strength and high stiffness. Between the ground level and Floor 13, where the floor slabs were assumed to have failed in Scenario A, the configuration and capacity of the lateral bracing was modeled by either a linear-elastic spring (labeled "LS" in Figures 5.16 and 5.17) or a nonlinear "Link" element (labeled "NLL" in Figures 5.17 and 5.18) with a defined axial-force/displacement relationship. Springs were assigned for header and seated type girder-to-column connections, on the basis that they had sufficiently high tension and compression capacities to brace the column. Nonlinear "Links" were defined for the welded double-angle knife girder-to-column connections. These links were assigned an unlimited compression capacity and a finite tension limit corresponding to their actual predicted tension capacity described below rather than their design capacity described in Section 4.2.5.

Although Section 4.2 demonstrates that the design tension capacity of the welded double-angle connection type, governed by flexural yielding of the angles per AISC, is insufficient to meet the code requirements for lateral bracing of Column 79, in reality, this type of connection has an even lower tension capacity. According to a detailed fracture analysis by Dr Anthony Ingraffea documented in Appendix A, a realistic estimation of the tension capacity of a typical welded double-angle connection, governed by fracture of the fillet welds, is approximately 0.85 kip/inch of connection depth, and possibly even lower.

The susceptibility of this type of connection to fracture under relatively low tension forces is a result of the stress concentrations created in the root of the fillet weld due to the combined tension and moment imposed on the fillet weld when the connection acts in tension (Figures 5.15 and 5.16). The tension capacity per unit depth determined by Dr Anthony Ingraffea was assigned to all nonlinear links representing knife connections in the Column 79 stability model because the details of these connections, including weld size and angle dimensions are similar throughout.

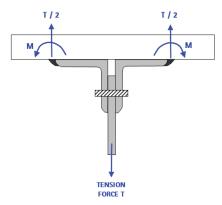


Figure 5.15 Combined tension and moment demands on fillet welds of knife connection due to tensile force

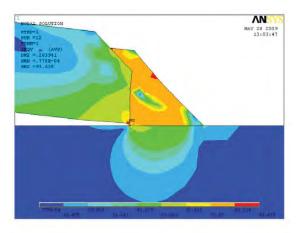


Figure 5.16 Stress concentration at root of fillet weld due to tension force on knife connection (image from report by Dr Anthony Ingraffea, Appendix A)

In the Scenario A stability analysis for Column 79, the load applied to the column corresponded to the sustained gravity load (1.0D + 1.0SDL + 0.25L) on the column from the undamaged global collapse model minus the loads lost from the floor collapse between Floor 13 and the ground.

The results of the Scenario A analysis, shown in Figure 5.17, illustrate that the western links representing the vulnerable double-angle knife connections framing into the western side plate of Column 79 reach their expected tension limit (ie fail) under sustained gravity loads. Once one link fails, the adjacent links above and below take more load and subsequently fail, causing an unzipping effect over the lower floors of the column. When a number of links have failed such that the minor axis of the column is unbraced over a sufficient height, the minor axis moments and deformations of the column increase exponentially, an indication of buckling. At a certain point, the combined effects of axial compression and bending moment on the column are sufficient to yield the column cross-section at its side plates. This point represents the buckling point and the end of the analysis because as soon as the side plates begin to yield and are unable to provide resistance to the bending forces inherent to buckling, the column loses stiffness. In other words, at the onset of yielding, a smaller section of the column must resist the same bending forces. This phenomenon results in the rapid deterioration of the stability of the column, or inelastic buckling (Ref 18). The nonlinear analysis does not account for the effect of residual stresses in the column which would only lead to an earlier onset of inelastic buckling.

In addition to the nonlinear analysis described above, a linear buckling analysis was also performed on the "unzipped" column to illustrate that the eigenvalue corresponding to the first buckling mode (or "buckling factor" in SAP2000) is less than 1.0 (ie unable to support 100% of or 1.0 times the applied load) and to further substantiate that the column would buckle under the sustained loads following the northeast corner floor failures.

The analysis for Scenario B was identical to that described above for Scenario A, except that the Link elements and the reduced vertical loads were assigned from the ground level to Floor 10 rather than to Floor 13. The results of the analysis, shown in Figure 5.18, also demonstrate that a northeast floor collapse between Floor 10 and the ground would cause Column 79 to buckle.

Figure 5.17 Input and Output for Scenario A Column 79 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

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Figure 5.18 Input and Output for Scenario B Column 79 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

Tribute Scritton         North         South         East           W144039         15         15         -           W144132         15         15         -           W144133         15         15         -           W144237         15         15         -           W144237         15         15         -           W144237         15         15         -           W144230         15         15         -           W144250         15         15         -           W144260         15         15<	W14459         LS         LS         LS           W144132         LS         LS         LS           W144132         LS         LS         LS           W144132         LS         LS         LS           W144133         LS         LS         LS           W144134         LS         LS         LS           W144135         LS         LS         LS           W14427         LS         LS         LS           W14427         LS         LS         LS           W14426         LS         LS         LS           W144260         LS         LS <th>W14429         LS         LS         LS         LS         Right Coll uniform Co</th> <th>  Northeast Comer   Northeast</th> <th>FRAME SECTIONS North North HARACING PROPERTIES IN THE MODEL  W14A32 I.S I.S W14A132 I.S I.S I.S I.S W14A133 I.S W14A232 I.S I.S</th>	W14429         LS         LS         LS         LS         Right Coll uniform Co	Northeast Comer   Northeast	FRAME SECTIONS North North HARACING PROPERTIES IN THE MODEL  W14A32 I.S I.S W14A132 I.S I.S I.S I.S W14A133 I.S W14A232 I.S
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## 5.5.2 Stage 2 Collapse Prevention

As described in the subsequent stages of the collapse sequence, it is the buckling of Column 79 in Stage 2 that transforms the collapse from a failure confined to the northeast corner of the building to a widespread collapse. The buckling of Column 79 due to its non-code compliant lateral bracing sets in motion a sequence of collapse events including additional interior column buckling, the failure of two transfer trusses, and ultimately the failure of the entire building.

An additional stability analysis was conducted on Column 79 to demonstrate that had the column been provided with code-compliant lateral bracing, it would not have buckled following the loss of the lower floors in the northeast corner of the building. The analysis used the same methodology described in Section 5.5.1 and Appendix C with the following exceptions. Rather than applying sustained loads, the full design load on the column, accounting for live load reduction, was conservatively applied. Also, rather than using the actual weaker capacities of the lateral bracing, the links were increased to provide either 1% or 2% of the design loads in each column at each level depending upon the number of sides on which the column was assumed to be braced. Figure 5.19 presents the principal parameters of the analysis as well as the results. Only Scenario A was considered because a demonstration of structural stability for Scenario A (ie floor failure initiation at Floor 13) establishes structural stability of Scenario B (ie floor failure initiation at Floor 10).

From the figure it is evident that after the application of the full design load on the column, no links have failed and as a result, the bending moments in the column are very low. The displacement plot shows nearly linear response up to the full axial design load of 7962 kips at Floor 7. The maximum displacement at the full design load is 0.005 in. In comparison, for the inadequate knife connection actually used, the displacement from Figure 5.17 increased to over 20in under the full sustained load. Therefore, the analysis confirms that had Column 79 been designed with 2% lateral bracing as required by the Building Code of the City of New York, Column 79 would not have buckled, and the collapse would have been arrested following the loss of the northeastern floor slabs below Floor 14 (or Floor 11) and the ground (ie Stage 1).

Figure 5.19 Input and Output for Stability Analysis for Column 79 (Scenario A) with 2% Bracing Capacity Illustrating Column Stability Under Sustained Gravity Loads

Non-vulnerable bracing direction Volnerable bracing direction 100 Cross Section: W14x730 + 2 Side Pl 28x3 Max Moment (btw Floor 7 & 8): 809.2 kip.in << Bending Capacity Max Shear Force (at Floor 7): 16.8 kip << 2% Capacity 80 LEGEND . 09 Displacement Plot at Floor 7 (minor axis) LOAD STEPS Northeast Corner Floor Collapse 40 COLUMN DOES NOT BUCKLE (E) H 20 9000 0.005 0.004 0.003 DISPLACEMENT (in) Bending Moment under Full Design Loads Shear Force under Full Design Loads Maximum Out of Plumbness: H/500 Initial Geometry Assumed per AISC tolerances Floor 46
Floor 45
Floor 43
Floor 43
Floor 43
Floor 40
Floor 40
Floor 40
Floor 30
Floor 30
Floor 35
Floor 35
Floor 36
Floor 36 Floor 20 Floor 19 Floor 17 Floor 16 Link Force under Full Design Load Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

COLUMN BARCHIE MOPERIES East/West Direction Link Force under Full Design Load North/South Direction REQ CONN AXIAL CAPACITY (2%) DESIGN AXIAL COLUMN LOADS [kip] W14x730 + 2 SIDE PI 26x1 W14x730 + 2 SIDE PI 26x2 W14x730 + 2 SIDE PI 26x2 W14x730 + 2 SIDE Pl's 26x3 W14x730 + 2 SIDE PI 26x2 W14x730 + 2 SIDE PI 26x3 W14x730 + 2 SIDE Pl's 26x4 FRAME SECTIONS W14x132 W14x370 W14x426 W14x455 FLOORS

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## 5.6 Probable Collapse Sequence Stage 3 Analysis Details

In Stage 3, Column 80 begins to buckle when the floors to its north and east start to fail as Column 79 buckles (Figure 5.5). Because Column 80, like Column 79, was already in a precarious state due to its insufficient lateral bracing (see Section 4.2), as soon as the floors supported by Column 79 begin to collapse, Column 80 loses stability. Consequently, the failure of Columns 79 and 80 and their tributary floors in Stages 2 and 3 occurs near-simultaneously, which is consistent with the formation of a vertical "kink" approximately along the lines of Column 79 and 80 in the video footage of the East Penthouse failure (Figure 5.1A). As Column 80 buckles, its tributary floor slabs, which are bounded almost entirely by trench headers, break off along the trench headers with minimal transfer of load to adjacent intact floors.

#### 5.6.1 Column 80 Stability Analysis

A nonlinear stability analysis was carried out in SAP2000 for Column 80 using the same methodology and assumptions described in Section 5.5.1 for Column 79. Nonlinear link elements were assigned over the height of the column to the south and west to represent the weak welded double-angle knife connections which provide the only lateral bracing following the loss of floor framing over the height of the column to its north and east as Column 79 fails. A tension capacity of 0.85 kip per inch of depth determined by Dr Anthony Ingraffea in Appendix A was assigned to all nonlinear links representing knife connections in the Column 80 model because the details of these connections, including weld size and angle dimensions, are similar to the connection that he analyzed. Because Column 80 begins to buckle immediately after Column 79, the gravity loads assigned to the Column 80 model at each floor level correspond to the original sustained loads (1.0D + 1.0SDL + 0.25L) on the column prior to the loss of the floor slabs supported by Column 79 to the north.

The results of the Column 80 stability analysis shown in Figure 5.20 are similar to those for Column 79. The western links representing the weak double-angle knife connections framing into the web plate of Column 80 begin to fail in tension under application of the sustained gravity loads. After a number of western links have failed between Floors 10 and 20, the minor axis deformations and moments in the column increase exponentially until the column cross-section begins to yield and the column is considered to have buckled. A linear buckling analysis was also performed on the same column with an unbraced length corresponding to the "unzipped" column height to illustrate that an eigenvalue (or "buckling factor" in SAP2000) less than 1.0 is produced for the sustained gravity loads on the column.

Figure 5.20 Input and Output for Column 80 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

			ilon	threction		Displacement Plot at Floor 13 (minor axis)	300	56	250	55	(in) 200	э ЛТ (	EWEN	⇒⇒ DAI9	S C C C C C C C C C C C C C C C C C C C	de	05	***		56					Cross Section:	Major Axis Moment Demand: 7197 kip-in	Minor Axis Moment Demand: 49674 kip-in	Governing Stress in Section:		Cross Section:	Major Axis Moment Demand:	Minor Axis Moment Demand:	H/W Maximum Stress in Section:		174	
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COLUMN BRACING PROPERTIES IN THE MODEL North South East West	W14x90 - NL(12.3kip) - LS	- NLL (12.3kip) - LS	W14x145 - NIL(12.3kip) - NIL(12.3kip) -	W14x193	- NILL(LZ:OKID)	- NIL (12.3kip) - LS	W14x283	W14000 - NLL(12.3kip) - LS	. VI 145.222 - NLL (12.3kip) - LS	- NLL (12.3kip) - LS	W/14~45.6 - NIL(12.3kjp) - L5	- NLL (12.3kip) - LS	- NLL (12.3kip) - LS	W14x550 - NLL(12.3kip) - LS	- NLL (12.3kip) - LS	W14x605 - NLL (12.3kip) - LS	W14x665 - NLL (12.3kip) - LS	- NLL (12.3kip) - LS	W14x730 - NLL(12.3kip) - LS	W14x730 + - NLL (12.3kip) - LS	- NLL(12.3kp) - LS	2 WEB P112.5x2 - NIL (12.3kip) - LS	W14x730 + NIL (12.3kip) - LS	- NLL (12.3kip) - LS	2 WEB PI 12:5x4 - NLL (12:3xip) - L5	W14x730 + - NLL(12.3kip) - LS	- NIL (12.3kip) - LS Nageria	2 WEB PI 12,5x6 - NLL (12,3kip) - LS Figure 1	W14x730 + - NLL (12.3kip) - LS	2 WEB PI 12.5x8 - NLL (12.3kip) - L5	W14x730 + - NIL(20.0kip) - LS		2 WEB P 12.5x8 - LS	M11. (23.0kip) NLL (23.0kip)	VV 4X/3U +	

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## **Guy Nordenson and Associates**

## 5.6.2 Stage 3 Collapse Prevention

Like for Column 79, a second stability analysis was conducted for Column 80 to demonstrate that had the column been provided with code-compliant lateral bracing, it would not have buckled following the loss of floors to its north and east as Column 79 buckled. The analysis used the same methodology and assumptions as the Column 79 analysis described in Section 5.5.2. Figure 5.21 presents the primary assumptions used in the analysis as well as the results. Similar behavior to the Column 79 is observed, indicating that the column would not have buckled had it been provided with the 2% lateral bracing required by the Building Code of the City of New York.

Non-vulnerable bracing direction 100 Vulnerable bracing direction W1 4x730 + 2Web P1 12.5x5 654 kip.in << Bending Capacity 11.5 kip << 2% Capacity LEGEND 650 LOAD STEPS Displacement Plot at Floor 13 (major axis) Cross Section: Max Moment (btw Floor 13 & 14): Max Shear Force (at Floor 13): Region of Floor Failure 40 COLUMN DOES NOT BUCKLE 20 0.6 0.5 0.4 0.4 0,1 0.9 0.8 Bending Moment under Full Design Shear Force under Full Design Loads 3 Ž Maximum Out of Plumbness: H/500 Assumed per AISC tolerances Hoor 44
Hoor 43
Hoor 45
Hoor 40
Hoor 40
Hoor 39
Hoor 36
Hoor 37
Hoor 37 Hoor 22
Hoor 21
Hoor 20
Hoor 19
Hoor 2 Link Force under Full Design Load Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link) COLUMN BRACING PROPERTIES Link Force under Full Design Load REQ CONN AXIAL CAPACITY (2%) 2204 2461 27.18 27.18 27.28 29.76 37.26 40.17 40 DESIGN AXIAL COLUMN LOADS [kip] W14x730 + 2 WEB PI 12.5x1 W14x730 + 2 WEB PI 12.5x2 W14x730 + 2 WEB PI 12.5x3 W14x730 + 2 WEB PI 12.5x4 W14x730 + 2 WEB PI 12.5x5 W14x730 + 2 WEB PI 12.5x6 W14x730 + 2 WEB PI 12.5x8 W14x730 + 2 WEB PI 12.5x8 W14x730 + 2 WEB PI 12.5x8 W14x730 + 2 SIDE PI 26x5 FRAME SECTIONS W14x550 W14x605 W14x665 W14x730 W14x500 W14x145 FLOORS

Figure 5.21 Input and Output for Stability Analysis for Column 80 with 2% Bracing Capacity Illustrating Column Stability Under Sustained Gravity Loads

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#### 5.7 Probable Collapse Sequence Stage 4 Analysis Details

A first principles calculation determined that it takes approximately 5 seconds for the upper floor areas supported by Columns 79 and 80 to reach the elevation of Transfer Trusses 1 and 2 at Floor 7. During this period, it is likely that the eastern-most diagonals of Transfer Trusses 1 and 2, which were located beneath these failing floors, were impacted by portions of the floors as they fell to the ground. Either the damage from these impacts accumulated in the diagonals and eventually led to their failure, or a small number of significant blows caused the diagonals to fail. The actual failure mechanism of the diagonals and the time required to fail them cannot be speculated due to the highly random nature of the event but has no impact on the overall analysis. Nevertheless, the 5 second period of time it takes for the upper floors of the building in the failing region to reach the elevation of the trusses is consistent with the 5 second pause in the video footage of the collapse following the fall of the East Penthouse and prior to the collapse of the remainder of the building.

The loss of the eastern diagonals of Transfer Trusses 1 and 2 leads to their complete failure and the collapse of Columns 76, 77 and 78 which are supported by the trusses Figure 5.6). The failure of Transfer Truss 1 is considered in greater detail in this section than the failure of Transfer Truss 2 because the failure of Transfer Truss 1 has negative consequences on the perimeter of the building as a result of the interdependency of the transfer systems. However, in both cases, the loss of the eastern diagonal of each truss results in the total failure of the truss, in the manner described below for Transfer Truss 1.

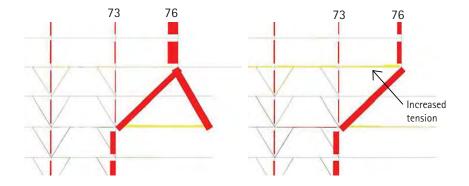
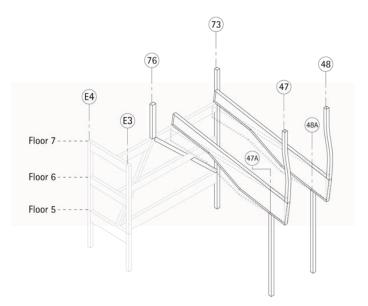


Figure 5.22 Transfer Truss 1 (view towards south) before and after east diagonal failure

The loss of the east diagonal of Transfer Truss 1 results in a redistribution of load to the remaining truss members, including a large tension force in the western half of the top chord of the truss, a W36x210 girder between Columns 73 and 76 at Floor 7. The axial force on this member changes from 30 kips in compression to 439 kips in tension following the loss of the east diagonal of the truss (Figure 5.22). According to the available construction documents this girder is non-composite, meaning that the tensile force cannot be redistributed into the concrete slab. While the girder itself is capable of supporting this tensile force, its connection to Column 73 is a 46.5"-deep welded double-angle knife connection with a maximum tensile strength of only 88 kips according to Dr Anthony Ingraffea's fracture analysis. The connection, therefore, fails under the increased load causing the girder to detach from Column 73. The loss of Girder 73-76 results in an unstable support condition for Column 76 on the remaining western diagonal of Transfer Truss 1. In this static analysis, the discounting of dynamic amplification effects due to the rapid failure of Transfer Truss 1 benefitted the performance of the structure.

Following the detachment of Girder 73-76 from Column 73, the two deep cantilevered transfer girders supporting Columns 47 and 48 on the north perimeter of the building begin to pivot about their lower supports (Columns 47A and 48A) due to the loss of the back-span support previously provided by the girder running between Columns 73 and 76 (Figure 5.23). The weight of the floor slabs framing into these girders and their continuity with the adjacent floor slabs to the west is not sufficient to prevent rotation of the girders.



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Figure 5.23 Rotation of Transfer Girders at Columns 47 and 48 following failure of top chord of Transfer Truss 1

The loss of back-span support eliminates the ability of the girders to act as cantilevers to transfer load from Columns 47 and 48 to Columns 47A and 48A. As a result, the north perimeter structure, including the belt truss and moment frames, must bridge over the two rotated transfer girders at Columns 47 and 48, resulting in increased loads to the perimeter supports immediately to either side of these members (Figure 5.24) and increased moment and axial demands on the perimeter frame.

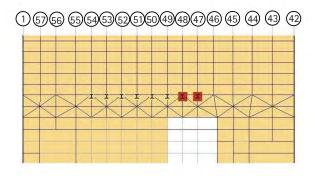


Figure 5.24 Bridging of north perimeter frame at Stage 4 between Columns 46 and 49 (failed transfer girders shown in red)

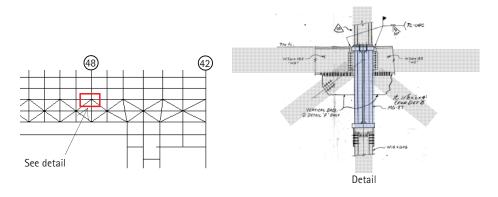


Figure 5.25 Detail from Frankel Steel erection drawing E119 showing discontinuity of perimeter belt truss members at connection to deep cantilevered girders (north elevation)

Additionally, because vertical and diagonal members of the perimeter belt truss system frame into the deep cantilevered transfer girders (Figure 5.25), the large rotations of these girders cause significant deformations in the belt truss system, including possible rupture of the connections to the transfer girders. In order to give benefit to the performance of the structure, this effect is not accounted for in the analysis.

The increase in load to the cantilevered girder supporting Column 49 to the west of the rotating girders is not large enough to fail either the adjacent transfer girder or Column 49/49A, which remains braced by interior floor slabs. However, the load shed to the perimeter structure frame to the east of the rotating girders has more severe consequences because Columns 44/44A, 45/45A and 46/46A are laterally unbraced about their minor axes as a result of the floor loss in the eastern region of the building. The axial load in Columns 44A, 45A and 46A increase by 8%, 18% and 52% respectively and these increases are sufficient to cause them to buckle. A linear buckling analysis on the global model at this stage of the collapse produces an eigenvalue, or "buckling factor" of 0.96 (ie less than 1.0) for sustained loads on the structure, indicating buckling of these columns. The buckling mode shape shown in Figure 5.26 illustrates that these columns buckle over the lower third of the building.

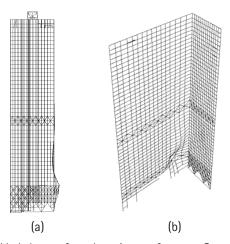


Figure 5.26 Buckled shape of north perimeter frame at Stage 4 in global model (a) view towards west (b) partial view towards northeast

As Columns 44, 45 and 46 lose the ability to support load, the north exterior moment frame and belt trusses must bridge an even greater distance between Column 49 and Column 42 (Figure 5.27). This behavior is represented in the global collapse analysis by the removal of Columns 44, 45 and 46 and their associated perimeter framing over the lower three floors of the building. As shown in Figures 5.28 and 5.29, the loss of

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Columns 44, 45 and 46 places additional demands on the members of the moment frame and belt trusses and causes the axial load in Column 42 to increase by 57% while it loses minor axis lateral bracing from the buckling north façade frame.

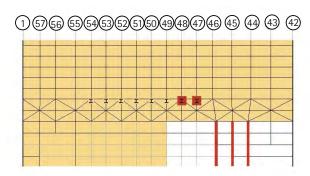


Figure 5.27 Bridging of north perimeter frame at Stage 4 between Columns 42 and 49 (failed transfer girders and buckled perimeter columns shown in red)

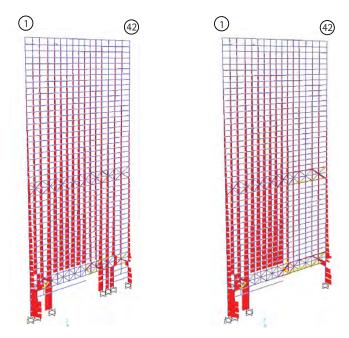


Figure 5.28 Redistribution of North exterior frame axial loads in Stage 4 due to buckling of Columns 44, 45 and 46

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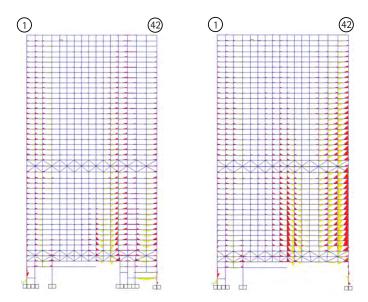


Figure 5.29 Increase in North exterior frame bending moments in Stage 4 due to buckling of Columns 44, 45 and 46

A linear buckling analysis of the global model at this stage provides an eigenvalue, or buckling factor, for sustained loads on the eastern perimeter frame of 1.23. A factor in this range is sufficient to indicate the likelihood of buckling due to the conservative nature of linear buckling analyses in general which assume ideal geometry and ideal material properties as well as this analysis in particular which used a lower-bound estimate of the loads present on the frame. Figure 5.30 shows the buckled shape of the eastern perimeter structure over approximately the lower half of the building.

The buckling of the northeast corner of the perimeter frame is likely assisted by the presence of the remaining damaged floor slabs in this corner of the building. While these floor slabs are expected to be too damaged by the deformations and hinge lines explained in Section 5.4.1 to provide lateral bracing to the corner of the building, it is likely that they create an interaction or coupling between the north and east perimeter frames of the building such that an out-of-plane buckling of the north perimeter frame over the lower floors of the building introduces torsion on the remaining slabs and transfers the deformations to the east perimeter frame. This effect is illustrated by the deformed shaped of a study model shown in Figure 5.31.

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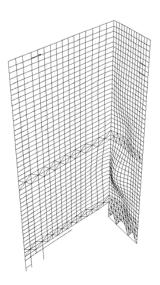


Figure 5.30 Buckled shape (factor=1.23) of east perimeter frame at Stage 4 in global model (partial view towards northeast)

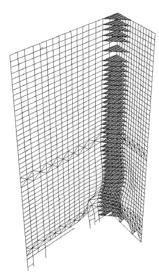


Figure 5.31 Buckled shape of a portion of a study model illustrating possible influence of corner floor slabs on the buckling behavior of the northeast corner of the perimeter frame

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#### 5.7.1 Stage 4 Collapse Prevention

Additional load path redundancy in the design of Transfer Trusses 1 and 2 would have allowed them to continue to carry load following the loss of their eastern diagonals and potentially preventing the failure of Columns 76, 77 and 78 which were supported by them. This redundancy could have been provided within each individual truss through the use of double X-bracing rather than inverted V bracing as well as between the trusses in the form of an additional North-South oriented truss.

Furthermore, the use of welded double-angle knife connections, which have minimal axial strength and stiffness in tension, should have been avoided in the vicinity of the transfer trusses which characteristically place axial demands on the connections between their chord members and supports. Even prior to the loss of the eastern diagonal of Transfer Truss 1, the axial force in Girder 76-73 was 32 kips, indicating that axial force transfer through these connections was high enough to warrant consideration in their design. Furthermore, these vulnerable connections reduced the ability of the trusses to redistribute load following the loss of their eastern diagonals.

In addition, had the transfer girders not been supported by Transfer Truss 1, its loss would not have set in motion the series of failures along the exterior of the building. Avoiding interdependency of transfer structures reduces the likelihood of disproportionate collapse with minimal impact on the programmatic requirements of the building.

#### 5.8 Probable Collapse Sequence Stage 5 Analysis Details

In Stage 5, simultaneous failures occur on both the interior and exterior of the building. Although the mechanisms responsible for the interior and exterior failures are independent, they occur simultaneously because they are both activated by the loss of the eastern floor slabs (Figure 5.7).

On the exterior of the building, the buckling of the northeastern perimeter of the building spreads to the south and west as load is redistributed. To represent the buckling of the northeast perimeter frame described in Stage 4, the lower three floors of Columns 35 to 42 and their associated perimeter framing were removed from the global model. A subsequent linear buckling analysis of the global model produced a SAP2000 "buckling factor" of 0.87 for the remaining gravity loads on the structure. The corresponding buckled shape in Figure 5.32 indicates that the remaining columns on the east perimeter of the building as well as Columns 24–27 on the south perimeter of the building have lost stability.

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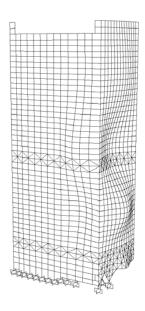
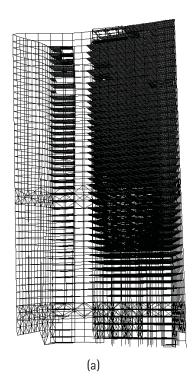


Figure 5.32 Buckled shape (factor=0.87) of east and south perimeter frame at Stage 5 in global model (partial view towards northwest)

The global model at this stage was also analyzed under the sustained gravity load case, and the resulting deformed shape is shown in Figures 5.33a and 5.34. These figures illustrate that the buckling of the lower northeastern corner of the perimeter frame in Stage 4 causes the unbraced eastern half of the building to sway northward. The deformed shape closely resembles the visual evidence of the "kink" in the north façade of the building immediately preceding total collapse of the building (Figure 5.33b).





(b)

Figure 5.33 (a) Deformed shape (not to scale) of global collapse model in Stage 5 following buckling of base of perimeter frame at the northeast corner (b) Comparison with deformed shape of building (ie horizontal "kink") just prior to total collapse

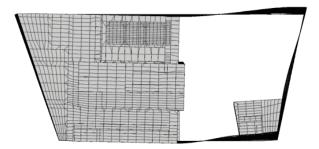


Figure 5.34 Deformed shape (plan, deformations not to scale) of global collapse model in Stage 5 following buckling of base of perimeter frame at the northeast corner

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At the same time that the perimeter buckling spreads to the south and west, additional collapse events occur on the interior of the building. The large tributary floor areas of Columns 76, 77 and 78 which had been supported by the transfer trusses fall simultaneously towards the ground. While the floor failures in the earlier stages of collapse could be localized by the presence of the trench headers bounding the failing floor areas, the floor areas tributary to Columns 76, 77 and 78 have greater connectivity to the adjacent structure. Furthermore, the mass of the large tributary areas is greater. As the eastern supports of these areas fail, each floor level rotates about its remaining western support, creating a centripetal force that imposes tension on the rotating floor (Figure 5.35). The horizontal component of this tensile force in turn applies an eastward pull on each floor to the west (Figure 5.36). A report on the diaphragm rupture analysis in Appendix D provides additional details on the estimated magnitude of these loads.

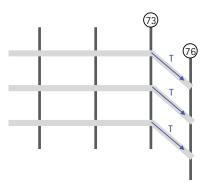


Figure 5.35 Basis for horizontal loading on western floor slabs

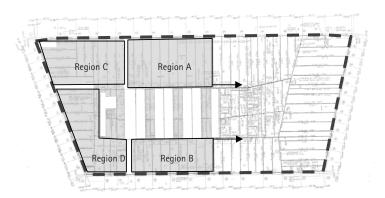


Figure 5.36 Horizontal loading on intact floor slab as floor areas tributary to Columns 76, 77 and 78 begin to collapse (perimeter moment frame shown with dashed line)

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The remaining western intact floor diaphragms are prone to rupture and instability from lateral loading due to the pre-segmentation by trench headers and core openings. The study detailed in Appendix D indicates that this vulnerability is the probable mode of failure of the remainder of the building.

It is probable that the horizontal force imposed by the collapsing structure on these floors fails the trench headers in tension, resulting in global instability and large displacements of portions of the diaphragm (Figure 5.37). As the western floor diaphragms begin to break apart at the trench headers and rotate horizontally, interior Columns 64 through 75, which are braced by these slabs, are compromised. These columns, which are already vulnerable to instability due to their numerous lateral bracing code violations, either break off from the rotating slabs and lose stability or remain connected to the rotating slabs and displace laterally. In either case, it is reasonable to conclude that these twelve columns buckle as a unit somewhere above Floor 7 where the interior core bracing stops and the columns are heavily loaded. This analysis is consistent with video footage showing the rapid fall of the visible roof structure supported by these columns immediately following the fall of the roof structure supported by Columns 76, 77 and 78 (Figure 5.1c and 5.1d).

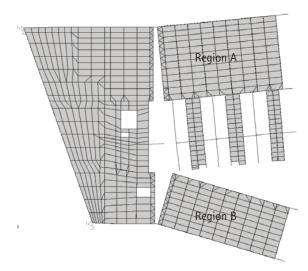


Figure 5.37 Rupture of trench headers from lateral loading leading to instability of Regions A and B of the diaphragm (deformations not to scale, failed links hidden from view for clarity)

At the same time, the loss of the floor areas tributary to Columns 76, 77 and 78 leaves Column 81 reliant on the minor axis lateral bracing provided by the vulnerable welded double-angle knife connection to its east. A nonlinear stability analysis on Column 81 with this bracing configuration using the same assumptions as the analyses on Columns 79 and 80 demonstrates that the column buckles to the west at this stage in the collapse (Figure 5.38). A second nonlinear stability analysis also demonstrates that had this column been provided with the code-required 2% lateral bracing, it would not have failed (Figure 5.39).

#### 5.8.1 Stage 5 Collapse Prevention

The manner in which the remaining intact floor areas rupture under lateral loads is a function of the discontinuities in the floor slabs created by the trench header ducts as well as the position of the lateral force-resisting systems in the building. The braced frames on the interior core stop at Floor 7. Above this level, the lateral force-resisting system consists only of a perimeter moment frame. As a result, the individual floor slab segments created by the trench headers and core openings are each only connected to the building's lateral system on their outside edge, and it is this asymmetry that leads to rupture under lateral loading. Had the braced frame at the core been extended through the entire height of the building, slab Regions A and B would have been braced symmetrically along both their north and south edges, and therefore would not have ruptured along the western trench headers. Additionally, had the discontinuities in the floor slabs created by the trench headers and core openings been addressed by the addition of horizontal bracing, the floor areas would not have ruptured in the manner they did.

Figure 5.38 Input and Output for Column 81 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

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Figure 5.39 Input and output for stability analysis for Column 81 with 2% bracing capacity illustrating column stability under sustained gravity loads

Non-vulnerable bracing direction Volinerable bracing direction 100 Cross Section: W14x730 + 2Web Pl 12.5x3.5 Max Moment (btw Floor 7 & 8): 831 kip.in << Bending Capacity Max Shear Force (at Floor 8): 12.0 kip << 2% Capacity 80 LEGEND 09 Displacement Plot at Floor 7 (minor axis) LOAD STEPS Region of Floor 40 ailure 81 COLUMN DOES NOT BUCKLE 20 0.02 0.01 0.005 0.045 0.04 0,035 0.03 0.025 DISPLACEMENT (in) Bending Moment under Full Design Loads Shear Force under Full Design Loads Maximum Out of Plumbness: H/500 Initial Geometry Assumed per AISC tolerances Hoor 46
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Hoor 47
Hoor 40
Hoor 38
Hoor 39
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Hoor 37
Hoor 37 Floor 21 Floor 19 Floor 18 Floor 17 Floor 16 Link Force under Full Design Load Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link) COLUMN BRACING PROPERTIES East/West Direction Link Force under Full Design Load REQ CONN AXIAL CAPACITY (2%) DESIGN AXIAL COLUMN LOADS [kip] -87.0 -176.2 -277.4 -404.8 -536.4 W14x730 + 2 WEB P1's 12.5x6.5 W14x730 + 2 WEB PI's 12.5x3.5 W14x730 + 2 WEB PI 12.5x1 FRAME SECTIONS W14x665 W14x730 W14x145 W14x176 W14x370 W14x426 W14x455 W14x605 W14x665 W14x730 W14x211 W14x257 W14x550 W14x99 FLOORS

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#### 5.9 Probable Collapse Sequence Stage 6 Analysis Details

In the final stage of the global collapse sequence, both the remaining interior and exterior structure of the building fail (Figure 5.8). When Columns 64 through 75 buckle and their floor areas collapse, they compromise five additional transfer girders supporting the north perimeter frame of the building. As a result, the only remaining vertical support for the north face of the building is provided by Columns 55, 54, 57 and 1. Similarly, on the south face of the building, the only vertical support is provided by Columns 15 to 23. In both cases, the perimeter moment frames and belt trusses above the zones where the perimeter structure has failed effectively cantilever from these remaining western supports. This behavior induces high bending moments in the frames and large axial forces in the belt truss diagonals, particularly in the areas closest to the remaining supports where force transfer is greatest (Figure 5.40). Plastic hinging of the moment connections and buckling of the belt truss diagonals is likely in this zone, and this behavior is consistent with the video footage showing localized façade failure in the region of Columns 53–55 on the north façade (Figure 5.41).

Simultaneously, the remaining vertical supports become overloaded and buckle. This behavior is confirmed by a linear buckling analysis of the global model at this stage following the removal of buckled Columns 24–27 and the floor structure tributary to Columns 64 to 75 including five transfer girders supporting the north facade. Numerous buckling modes are found corresponding to the failure of the remaining northern and southern supports over the lower floors of the building (Figures 5.42 and 5.43), an indication of the consequent total collapse of the building.

The timing of the identified key events in the video footage indicates that the spread of exterior frame buckling from the east side of the building to the west described in Stages 4–6 occurs in a very short period of time. This rapid failure is consistent with the fact that the roof of the building remains virtually horizontal as it collapses (Figure 5.1d).

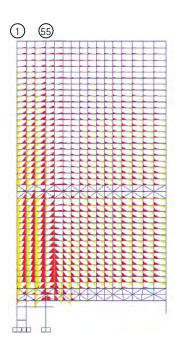


Figure 5.40 Bending moment diagram of north façade in Stage 6 indicating perimeter frame failures in the region of Column 55



Figure 5.41 Still from available video footage of final stage of collapse showing window breakage in the region of Columns 53-55 on the north elevation, possibly an indication of plastic hinging of the moment frame

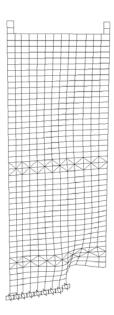


Figure 5.42 Buckled shape of south perimeter frame in Stage 6 of the global Model (looking northeast)

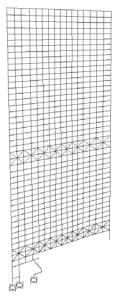


Figure 5.43 Buckled shape of north perimeter frame in Stage 6 of the global Model (looking northeast)

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#### 6.0 SUMMARY OF FINDINGS

This section provides a summary of findings regarding the probable cause of the global collapse of the World Trade Center 7 office building on 11 September 2001 and the inherent vulnerabilities of the structure which made it susceptible to disproportionate collapse:

- The failure of a single girder due to fire effects at Column 79 resulted in the complete global collapse of the building.
- Debris damage had no influence on the global collapse of the building.
- The office contents fires present in the building at the time of the collapse, while having an influence on the initiating local collapse event, were not the cause of the global collapse of the building.
- Inadequate lateral bracing of the building's interior columns was the cause of the progression of global collapse. Had the interior columns been provided with the code-required lateral bracing, global collapse would not have occurred.
- According to the contract documents, the design team did not consider the 2% lateral bracing requirements of Section C26-100.2 of the Building Code of the City of New York in the design of the building.
- The welded double-angle knife connections that connected over half of the girders and beams to interior columns were inherently weak in tension and not adequate as lateral bracing. Simple hand calculations would have demonstrated that it was not possible to design an AISC-compliant welded double-angle knife connection for the tensile forces required to provide the 2% lateral bracing to the heavily-loaded columns in the WTC7 structure.
- Eighteen of the twenty-four interior columns and 46% of the floor-to-interior column joints violated the lateral bracing requirements of the code.
- The actual tensile capacity of the welded double-angle knife connections was even weaker than the AISC code predicted due to their susceptibility to weld fracture.
- The use of multiple interdependent transfer structures contributed to the spread of disproportionate collapse. Had these transfer structures been made more redundant and had there been less interdependency of the transfer systems, it is probable that the global collapse would not have progressed west of Columns 79 and 80.

- Other characteristics of the building made it particularly susceptible to disproportionate collapse including the large tributary floor areas of interior columns and the trench headers subdividing the floor slabs. These characteristics explain the way in which the building failed.
- The probable global collapse sequence detailed in Section 5.0 is consistent with the visual evidence of the collapse on 11 September 2001.
- Although the global collapse occurred in the particular sequence outlined in Section 5.0, it is my opinion that disproportionate collapse could have resulted from an initiating failure at numerous other locations in the building as a result of the pervasive lateral bracing code violations in the building and the other vulnerabilities outlined in this report.
- The analyses performed to support the above conclusions were straightforward, transparent and conservative.

#### 7.0 REFERENCES

- 1 American Concrete Institute Committee 318, *Building Code Requirements for Structural Concrete (ACI 318–05) and Commentary (ACI 318R–05)*, Second Printing, American Concrete Institute, Farmington Hills, MI, August 2005
- 2 American Institute of Steel Construction, Inc (AISC), *Manual of Steel Construction*, Eighth Edition, 1980
- 3 American Institute of Steel Construction Inc (AISC), Steel Construction Manual, Thirteenth Edition, 2005
- 4 American National Standard Institute (ANSI), *Minimum Design Loads for Buildings and Other Structures*, 1982
- 5 American Society of Civil Engineers (ASCE), Seismic Rehabilitation of Existing Building, ASCE Standard ASCE/SEI 41-06, 2007Anderson T, Fracture Mechanics: Fundamentals and Applications, 3<sup>rd</sup> Edition, CRC
- 6 Anderson T, Fracture Mechanics: Fundamentals and Applications, 3<sup>rd</sup> Edition, CRC
- 7 Bailey, Colin, Expert Report: Response of WTC7 to Standard Office Fires and Collapse Initiation, February 2010
- 8 Building Code of the City of New York, 1979-1980
- 9 Calixto J, "Comparative Study of Longitudinal Shear Design Criteria for Composite Slabs", *Ibracon Structures and Materials Journal*, June 2009
- 10 Cedolin L, Bisi G, Nardello P, *Mode II Fracture Resistance of Concrete*, Concrete Science and Engineering, Vol 1, March 1999, pp 37-44
- 11 ICC Evaluation Service Inc, ICC ES Legacy Report on HH Robertson Steel Floor Deck (ER-2739), 1996
- 12 Lam D and Qureshi J, "Prediction of Longitudinal Shear Resistance of Composite Slabs with Profile Sheeting to Eurocode 4", *The Regency Steel Asia International Symposium on Innovations in Structural Steel (RSA-ISISS)*, 2008
- 13 Luttrell, Larry, Steel Deck Institute (SDI) Diaphragm Design Manual, Second Edition

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- 14 Mowrer, Fred, Expert Report in re: September 11 Property Damage and Business Loss Litigation, February 2010
- 15 McGuire, William, *Steel Structures*, Prentice-Hall International Series in Theoretical and Applied Mechanics, Structural Analysis and Design Series, 1968
- 16 Rogers C, Hancock G, *Tensile Fracture Behaviour of Thin G550 Sheet Steels: Research Report No R773*, University of Sydney Department of Civil Engineering, 1998
- 17 SAP2000 Nonlinear Version 12.0.2, Computers and Structures, Inc., Berkeley CA
- 18 Yura, Joseph, Five Useful Stability Concepts

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APPENDIX A

WTC7 Knife Connection Report By Anthony Ingraffea PE PhD

WTC7 Global Collapse Analysis Report and Summary of Findings – Appendix A 12 February 2010

# WTC7 Knife Connection Study Report

Submitted to
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by

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November 30, 2009

#### 1.0 INTRODUCTION

The purpose of this report is to investigate the capacity of a welded double-angle connection of the type shown in Figure 1, hereafter called a "knife" connection. This type of connection was used to connect many girders and beams to interior columns in the World Trade Center 7 building (WTC7) which collapsed on September 11, 2001. These connections were designed for transfer of vertical shear load, by longitudinal loading of the fillet welds; however, in supplying lateral restraint against buckling of columns, they would also have to transmit direct tension load, by transverse loading of these welds. The specific geometry of the connection shown in Figure 1 corresponds to the south knife connection to Column 79 at Floor 13 (according to Frankel Steel Limited Drawing No. 1091, Rev May 23 1985). This specific detail was used in numerous other locations in the building for connections to interior columns.

In Section 2, this report first describes the state-of-the-practice approach to calculating the capacity of this type of connection, based on AISC recommended practice. In Section 3, it presents a state-of-the-art, non-linear fracture mechanics approach to predicting the capacity of connections with low-eccentricity, transversely loaded welds. In Section 4, this approach is applied to a specific instance of the connection type shown in Figure 1b, a connection to Column 79 in the WTC7 building. A comparison between capacity predictions based on AISC recommended practice and the non-linear fracture mechanics approach is presented in Section 5.

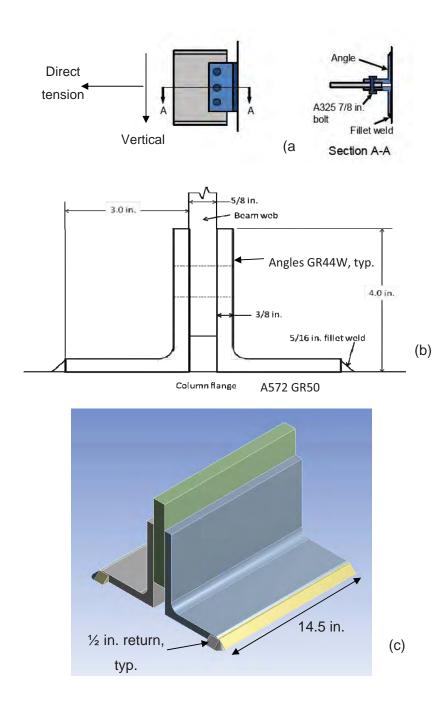


Figure 1. (a) Schematic of typical beam-column knife connection. (b) Column 79 knife connection in WTC7, cross-section. (c) Column 79 knife connection in WTC7, perspective (dimensions based on Frankel Steel Limited Drawing No. 1091, Rev May 23 1985)

#### 2.0 STATE-OF-THE-PRACTICE APPROACH

Figure 2 shows some typical welded details in which the weld is loaded transversely. In such configurations, the weld is situated at the tip of a crack-like root notch. Kanvinde (2009) note that many previous studies have shown that "...transversely loaded fillet welds are 50% stronger than longitudinal welds..." and a simplified strength relationship based on weld metal ultimate strength, F<sub>u</sub>, was "...adopted by CSA-S16 [17] in 1989 and was later presented in Appendix J of AISC [18], to be adopted in the main body of the specification in 2005, AISC [12]."

$$P_u = 1.5 \times 0.6 \times F_{u,weld} \times A_{throat}$$
 (1)

where

$$A_{\text{throat}} = L_{\text{weld}} \times \frac{1}{\sqrt{(1/L_{\text{shear}})^2 + (1/L_{\text{tension}})^2}}.$$

and "...the 1.5 factor reflects the 50% increase in strength for the transverse welds, while the 0.6 factor relates the axial strength to shear strength" according to the von Mises plasticity model.

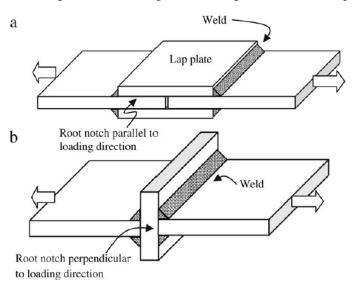


Fig. 1. Transverse welds in (a) Lap-welded specimens and (b) Cruciform type specimens.

Figure 2. Figure 1 from Kanvinde (2009): Typical details with low-eccentricity, transversely loaded welds.

For the detail shown in Figure 1(b) and (c),  $L_{shear}$  and  $L_{tension} = 0.3125$  in., and for each inch of weld, and for two welds,

$$A_{throat} = 2 \text{ x } 1.0 \text{ inch x } 0.22 \text{ inch} = 0.44 \text{ in}^2$$

Therefore, for each inch of weld in the WTC7 knife connection, the capacity for transverse loading only is

$$P_u = 1.5 \times 0.6 \times 0.44 \text{ in}^2 \text{ X } F_{u,weld} = 0.40 \times F_{u,weld} = 0.40 \times 70 \text{ ksi} = 27.7 \text{ kips}$$

Note that this is the capacity per inch of connection depth, as it accounts for both welds. Similarly, hereafter all predicted capacities are per inch of connection depth.

For longitudinal loading alone, the capacity would be  $V_u$ = 27.7 kips/1.5 = 18.5 kips per inch of connection depth. For both transverse and longitudinal loads, the accepted interaction equation is:

$$1 \ge (V/V_u)^2 + (P/P_u)^2 \tag{2}$$

where

V = applied shear load/inch of weld

V<sub>u</sub> = longitudinal capacity/inch of weld

P = allowable transverse load/inch of weld

In the present case, under 1.0D + 1.0SDL + 0.25L, the knife connection must transmit 41.3 kips of vertical shear. Equation 2 yields a transverse load capacity only slightly less than that for transverse loading alone,  $27.4 \, kips/inch$  of connection depth

[Note: a check of other possible failure mechanisms for such a connection would show that failure due to combined load yielding of the welds would not be the governing mechanism. Rather, the governing mechanism would be yielding of the angles, which would occur at about 1.4 kips per inch of connection depth.]

It should be noted that this capacity according to Equation 1 is independent of the toughness of the weld material, and also independent of the eccentricity of the transverse load to the weld line in the detail. Note also that this value assumes that the full strength of the weld material can be mobilized *before its ductility is exhausted*. If the elasto-plastic stress and strains at the front of the crack-like root notch reach a critical value before the ultimate strength is attained across the weld throat, the weld will fracture at a load less than that predicted by the AISC equation (1). This possibility was recently examined, in connection details with low eccentricity, by Kanvinde *et al.* (2009). Their alternative approach is presented next.

#### 3.0 NON-LINEAR FRACTURE MECHANICS APPROACH

Two timely and highly relevant studies on capacity of transversely loaded weld details are Kanvinde  $et\ al.$  (2009) and Kanvinde  $et\ al.$  (2008). These experimental and computational studies address the problem of fillet welds loaded transversely to the weld axis, using the cruciform configuration shown in Figure 2b. In such configurations, Kanvinde (2009) note that the weld is situated at the tip of a crack-like root notch. The key question governing weld strength is whether there is ductile mobilization of ultimate strength of the weld material before it exhausts its fracture toughness, in the form of the non-linear fracture mechanics parameter,  $J_{Ic}$ .

Kanvinde (2009) performed 24 tests, varying weld electrode type (E70T7 and E70T7-K12 enhanced toughness, both Grade 480 MPa, the same strength used in the WTC7 knife connections), weld size (8 and 16 mm), and notch length 32 and 64 mm. Significantly, Kanvinde also described a general approach, based on non-linear fracture mechanics computation and testing, that can be applied to other configurations with larger eccentricity.

There are three significant observations resulting from these recent Kanvinde papers:

1. The state-of-the-practice, represented by equation 1, appears to work well for connections of the type shown in Figure 2. However, doubt is cast on the applicability of this equation for connections with "...notch lengths significantly larger than those tested...", and on connections using non-toughness rated welds. The WTC7 connections had notch lengths significantly larger than those tested by Kanvinde, and, being pre-Northridge designs, likely did not use modern toughness-rated welds.

- 2. The non-linear fracture mechanics approach used by Kanvinde is the state-of-the-art technique for predicting connection capacity when it is limited by weld toughness. Both material and geometric non-linearity are included in their 2D finite element calculations of crack driving force, J<sub>I</sub>. This approach is used in the present investigation, in both 2D and 3D. This approach permits generalization to connection geometries significantly different than that shown in Figure 1b, and including shear as well as tensile loading on the weld.
- 3. Kanvinde, and many other investigators, point out the substantial difference in toughness that might exist among various weld electrodes. The types tested by Kanvinde met or closely approximated the post-Northridge requirement of 20 ft-lb (impact CVN value) at 21°C. This observation led to the sampling, presented later herein, of results from post-Northridge investigations of electrode toughnesses.
- 4. The non-linear fracture mechanics-based predictions from Kanvinde correlated well with the AISC strength prediction for the connection types shown in Figure 2.

#### 4.0 PRESENT INVESTIGATION OF WTC7 KNIFE CONNECTION

This section follows up on the Kanvinde investigation by applying the non-linear fracture mechanics approach used therein to the particular case of the high-eccentricity WTC7 knife connection. All of the observations and conclusions in Kanvinde (2009) are based on testing and simulation of the cruciform test configuration, Figure 1b. This configuration, although providing transverse loading to the weld, does not replicate the prying action of the outstanding leg seen in the case of the knife connection. It does not reproduce the high ratio of bending stress to normal stress across the weld which obtains in the knife connection, and it does not reproduce the additional prying action caused by vertical shear of the weld. Certainly, previous testing on the lap weld configuration (Figure 1a) is even more dissimilar to the knife connection. Consequently, the J-demand curves produced in Kanvinde (2009), Figure 3, might not be applicable to the knife connection. Therefore, the following actions have been taken:

- 1. Perform preliminary, FRANC2D elasto-plastic (small displacement) analyses on a 2D cross-section of the knife connection (based on Frankel Steel Limited Drawing No. 1091, Rev May 23 1985);
- 2. Perform ANSYS elasto-plastic (large displacement) analyses on a 2D cross-section of the knife connection;
- 3. Survey literature for toughness values of various pre- and post-Northridge electrodes; and

4. Perform ANSYS elasto-plastic (large displacement) analyses on a fully 3D model of the knife connection (based on Frankel Steel Limited Drawing No. 1091, Rev May 23 1985), including the effects of shear load, weld return, and load order effects.

# 4.1 FRANC2D Elasto-plastic (small displacement) Analyses on a 2D Cross-section of the Knife Connection

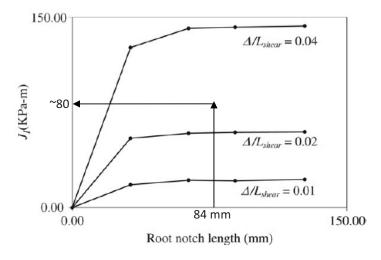
FRANC2D (Bittencourt *et al.*, 1996) was used to compute load-displacement curves, elastoplastic stress fields in the connection, especially in the weld area,  $\Delta/L_{shear}$  values, and  $J_I$  values for comparison to  $J_{Ic}$  values obtained by Kanvinde (2009), Table 1. These computations were exploratory, used to create an experience base and to form bases for verification of later 2D and 3D analyses.

Note that the  $J_{Ic}$  value measured by Kanvinde and most likely an upper bound to the WTC7 knife connections is that for the E70T7, 8 mm weld, 145 kPa m (0.83 k/in). Note also that this calibrated value is only about 35% of the toughness of the same size weld in the enhanced toughness weld material. Kanvinde (2009) defines a particular, specimen dependent, weld elongation measure,  $\Delta/L_{shear}$ , shown in Figure 4, as a metric for weld ductility. A similar definition is used for the FRANC2D results reported herein.

For all of the FRANC2D results reported herein, the following conditions prevailed:

• E = 29,000ksi, Poisson's ratio = 0.3, von Mises isotropic strain hardening constitutive model (same used in Kanvinde (2009)), with F<sub>y</sub> = 50 ksi in the angle and column materials. This yield strength, rather than angle design strength of 44 ksi, was assumed because 49.6 ksi corresponds to the average yield strength of CSA G40.21-44W steel based on the available mill test reports (refer to Section 3.4.1 of GNA main summary report). F<sub>u</sub> = 77 ksi was used in the weld material. (Kanvinde (2009) measured a mean value of 76 ksi on E70T7 filler metal, Table 2; 77ksi also corresponds to the use of an expected strength factor of 1.1 on the design strength of 70ksi, which is prescribed by ASCE/SEI 41-06 (refer to Section 3.4.1 of GNA main summary report). A strain hardening modulus of 100 ksi was used. Kanvinde (2009) did not report the stress strain

- curves from their tests. This modulus was varied by a factor of ten, up and down, with only about a 10% change in key output values in FRANC2D.
- FRANC2D uses the standard incremental-iterative technique during elasto-plastic analysis. Loads were applied in 10 equal increments, and a tolerance of 0.0005 on both displacement and residual load was used.
- In FRANC2D, a crack tip in an elasto-plastic material is surrounded by a symmetric template of collapsed Q8 elements with uncoupled crack tip nodes, Figure 5b. This is a standard technique to represent crack tip blunting under infinite strain with finite stress. All other elements are either standard Q8 or T6 types, Figures 5a.
- Half-symmetry was used on a plane strain cross-section of the knife connection. Also, a
  portion of the column flange was added so that an accurate representation of the fields in
  and around the weld could be obtained.



**Fig. 13.** Schematic plot showing the dependence of the toughness demand (*J*-integral) on root notch length for various deformations.

Figure 3. Figure 13 from Kanvinde (2009). J-demand relationship for cruciform connections with low eccentricty. Arrows and their values added herein.

Table 1. Table 5 from Kanvinde (2009)

**Table 5** Calibrated  $J_{IC}$  values for different weld sizes and classifications

Weld classification	Weld size (mm)	Average $J_{IC}$ (kPa m)	COV
E70T7	12	205	0.21
E70T7-K2	8 12	145 406	0.21 0.19
	8	417	0.24

Table 2. Table 1 from Kanvinde (2009).

**Table 1**Results from tension tests and Charpy V Notch Tests

Filler metal	Test	Tension tests			CVN energy (	CVN energy (J)		
		F <sub>y</sub> <sup>a</sup> (MPa)	$F_u^b$ (MPa)	$d_o/d_f$	$\varepsilon^{c}$	−29 °C	21 °C	100°C
E70T7 (Grade 480)	1	523	669	1.35	0.60	7.5	25.8	55.6
	2	530	670	1.15	0.28	8.1	24.4	55.6
	Mean	526	670	1.25	0.45	7.8	25.1	55.6
E70T7-K2 (Grade 480)	1	570	672	1.65	1.00	40.7	75.9	119.3
	2	572	672	1.74	1.11	31.2	84.1	119.3
	Mean	571	672	1.69	1.05	35.9	80.0	119.3

<sup>&</sup>lt;sup>a</sup> Measured yield stress, based on 0.2% offset method; static value.

 $<sup>^{</sup>c}$   $\varepsilon = \ln \left(d_{0}/d_{f}\right)^{2} = \text{average true strain across necked cross section of tension coupon.}$ 

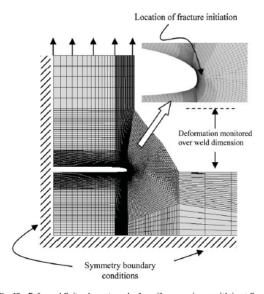


Fig. 12. Deformed finite element mesh of cruciform specimen, with inset figure showing a magnified view of the equivalent plastic strain contours in the notch tip region.

Figure 4. Figure 12 from Kanvinde *et al.* (2008), showing the definition of the weld deformation,  $\Delta/L_{\text{shear}}$ .

Measured ultimate strength; static value.

The results of the FRANC2D analyses are as follows:

**Deformed shape of knife connection at maximum load:** Figure 5c shows the predicted displaced shape (without amplification) at a predicted load of 3.25 kips/inch of weld (for both welds). Clearly, at this load level the deformations are large enough to cause some error in the field results due to the small displacement assumption used in FRANC2D. The last of ten load increments required about 3000 iterations to converge to this load, and no convergence was achieved at a load level higher than 4.38 kips/inch of weld. Figure 6 shows the FRAN2D-predicted load-displacement relationship, with a capacity of 3.25 kips/inch of weld indicated, based on a suspected upper-bound value of J<sub>Ic</sub> of 0.83 kips/inch.

**Stress and Strain fields:** Figure 7a shows contours of y-stress component at a load of 3.25 kips/inch of weld. It can be seen in this figure that this stress in the vicinity of the crack tip has substantially exceeded weld material  $F_u$ . Kanvinde (2009) measured a mean  $F_u = 97$  ksi on E70T7.

Figure 7b shows contours of von Mises effective stress at a slightly higher load of 3.75 kips/inch of weld. This effective stress combines all the stress components active in a non-uniaxial situation, like that in effect here. According to the von Mises yield criterion, most commonly applied to structural steels and their weld materials, the local yield strength of such materials depends on the local multiaxial stress state. Yielding occurs when the von Mises effective stress reaches the yield strength of the material in uniaxial tension, F<sub>y</sub>. The contours in Figure 7b show that the effective stress is substantially above uniaxial yield, 77 ksi here, and above F<sub>u</sub> on some sections, along the double-arrowed line emanating from the crack tip and terminating at the weld surface.

Taken together, these 2 predictions indicate that the weld has fully yielded and begun to substantially strain harden at a load between 3.25 and 3.75 kips/inch of weld, and, in a non-fracture mechanics sense, also begun to fail from the crack front.

Fracture mechanics parameters: As noted in Table 1, above, Kanvinde (2009) calibrated inelastic fracture toughness values,  $J_{IC}$ , through a combination of physical testing and finite element analyses of the test configurations. Using additional finite element analyses, they extrapolated their predictions through a set of J-demand versus normalized weld deformation curves shown in Figure 3. Figure 8 shows the FRANC2D-predicted normalized weld deformation plot for the knife connection, and indicates a peak value for this geometry of about 0.025. Using this value and the notch length in the knife connection of about 84 mm leads to a J-demand of about 80 kPa-m (0.46 kips/in). FRANC2D does not have the capability to directly compute J; however, it can compute crack-tip-opening-displacement (CTOD,  $\delta_t$ ). A well-known, approximate relationship, based on empirical testing and finite element analysis, between J and  $\delta_t$  is

$$J = M \times F_{v} \times \delta_{t} \tag{3}$$

where M is a dimensionless constant which varies between 1.15 and 2.95, with a generally accepted value of about 2 for moderate strength steels. FRANC2D predicts the load vs.  $\delta_t$  relationship shown in Figure 9, with a  $\delta_t = 0.0057$  inch at a load of 3.25 kips/inch of weld. Consequently, at this load FRANC2D predicts  $J_I = 0.88$  kips/in (171 kPa m), close to the demand predicted in Figure 3 and the critical value,  $J_{Ic}$ , of 0.83 kips/in (145 kPa m) measured by Kanvinde (2009) for E70T7 weld metal, Table 2. This fracture-mechanics-based failure prediction is wholly consistent with the failure prediction based only on observation of the stress fields, cited above, and seen in Figure 7.

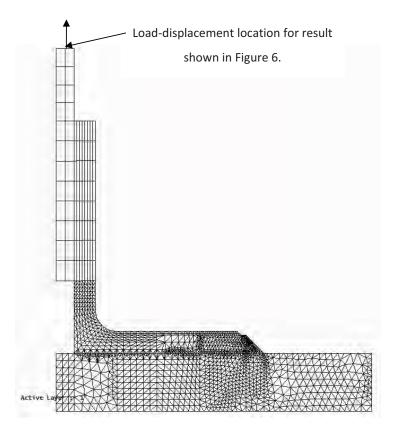


Figure 5a. Un-deformed FRANC2D mesh of WTC7 knife connection using symmetry.

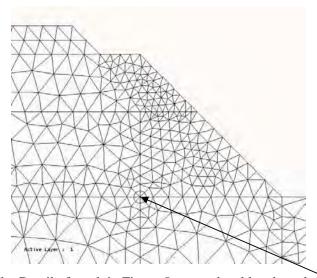


Figure 5b. Detail of mesh in Figure 5a around weld and crack front.

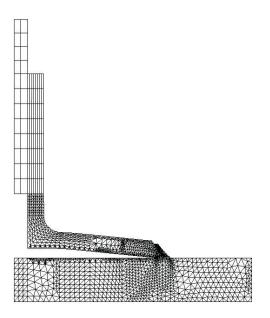


Figure 5c. FRANC2D-predicted deformation at load = 3.25 kips. Prying action on the fillet weld evident. No amplification.

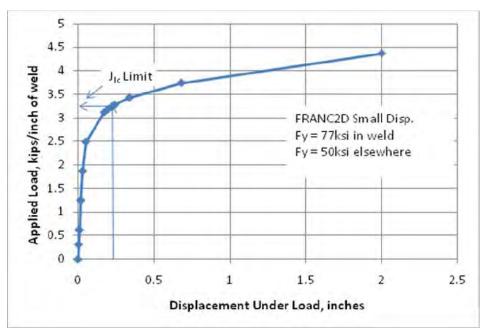
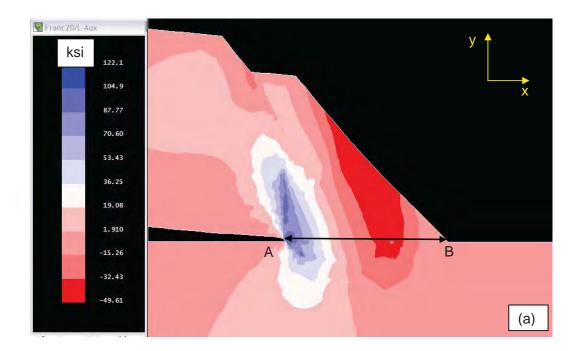


Figure 6. Predicted load-displacement relationship from FRANC2D, small displacement theory. The  $J_{IC}$  limit shown is for a suspected upper-bound value of 0.83 kips/in obtained by Kanvinde (2009) for E70T7 weld filler. No shear force included.



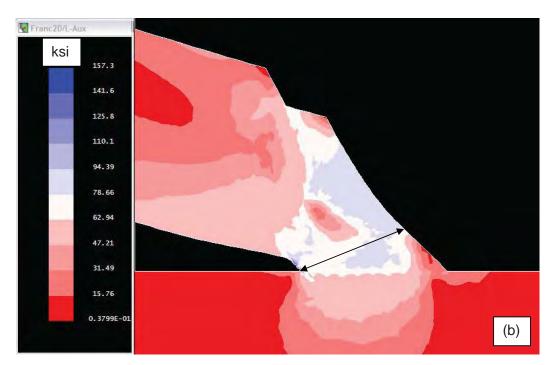


Figure 7. (a) FRANC2D-predicted y-stress contours at a load of 3.25 kips/inch of weld; (b) effective stress at a load of 3.75 kips/inch of weld on deformed shape (magnification = 1).

These exploratory results from a FRANC2D study of the knife connection indicate that the AISC equation substantially over-predicts weld capacity in the WTC7 knife connection: 27.4 versus 3.25 to 3.75 kips per inch of weld. The most likely reason for this over-prediction is that the AISC equation assumes no bending in the weld. Prying action arising from high eccentricity is not acknowledged: the throat area is assumed to be uniformly loaded to F<sub>u</sub>. Although the cruciform configuration used in the Kanvinde studies creates some eccentricity on the weld, it does so only by increasing the thickness of the loaded plate: this results in a much more direct load path through the weld with no prying action and, therefore, a low ratio of bending to normal stress across the weld. For example, Figure 10a shows a FRANC2D finite element model of one of the cruciform details tested and analyzed by Kanvinde. Figure 10b shows contours of x-component stress in a weld region under a load of 5 kips per inch of weld on the displaced shape at a magnification factor of 100. This figure shows very low levels of crack opening, i.e. no prying action, and stress levels below yielding, even at the crack front. Figure 11a shows the FRANC2D-predicted distribution of x-stress along a radius emanating from the crack tip and terminating at the weld toe in Figure 10b. This plot is another indication of a low level of eccentricity in that the distribution is entirely tensile.

In contrast, Figure 11b shows the FRANC2D-predicted distribution of y-stress along a radius emanating from the crack tip and terminating at the weld toe in the WTC7 connection. The effect of bending in the weld is clearly evident, as over one-half of this radius is in compression. Entirely unlike the cruciform connection, in the knife connection the weld capacity is limited by the *difference in the force resultants* computed from the tensile and compressive areas shown in this figure: for a given  $F_u$ , load capacity of the weld is proportional to difference between area in tension (T) and area in compression (C).

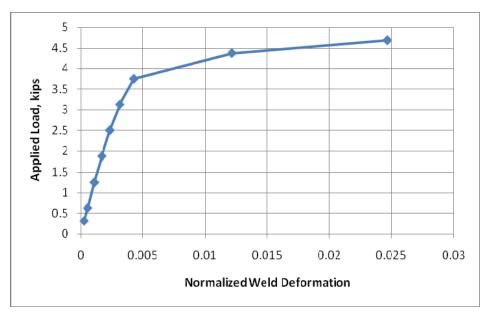


Figure 8. Predicted load versus normalized weld deformation,  $\Delta/L_{shear.}$  From FRANC2D with small displacement theory.

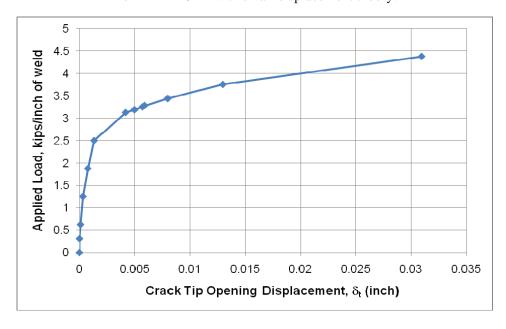


Figure 9. Predicted load versus crack tip opening displacement,  $\delta_{t.}$  From FRANC2D with small displacement theory.

# 4.2 ANSYS Elasto-plastic (large displacement) Analysis on a 2D Cross-section of the Knife Connection

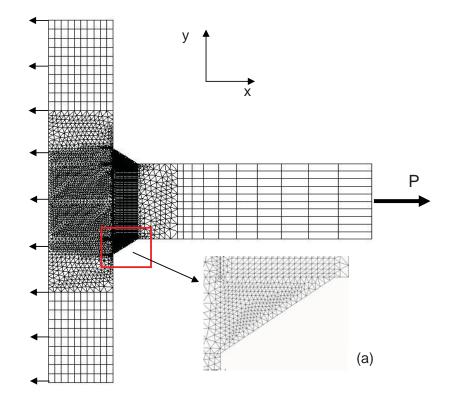
ANSYS has large displacement capability. Consequently, it was used to follow up on the FRANC2D calculations both to verify and to extend these to the large deformation regime. In all the 2D ANSYS analyses used herein, the same types of elements as those used by FRANC2D were employed. However, ANSYS does not use collapsed Q8 elements around the crack tip, but does compute values of  $J_I$  at a crack tip directly rather than using the indirect method involving the intermediate calculation of  $\delta_I$ .

Figure 12 shows the ANSYS-predicted load-displacement plot. At a load level of 4.7 kips/inch of weld, ANSYS predicts a  $J_{\rm I}=0.83$  kips/in (145 kPa m), the upper-bound critical value,  $J_{\rm Ic}$ , measured by Kanvinde (2009) for E70T7 weld metal. The von Mises effective stress distribution in the weld at this load level is shown in Figure 13. These stress results show consistency with those from FRANC2D; however, there is a significant difference in the predicted load-displacement plots due to the large deformations involved. Therefore, ANSYS large deformation capability will be used in the 3D calculations to follow.

## 4.3 Information on Toughness Values for Weld Materials

The value of  $J_{IC}$ , 0.83 kips/in, determined by Kanvinde for post-Northridge E70T7 electrode is likely an upper-bound on the toughness in the WTC7 knife connection. Tables 3 and 4 contain toughness data in 3 forms for selected pre- and post-Northridge weld electrodes of the E7X series (7X ksi yield strength). In Table 3, the original source data is in the form of impact CVN values (in red). Empirical conversions from impact CVN values to  $K_{Ic}$  values are given in Barsom and Rolfe (1987). For the transition region of the CVN data,

$$K_{Ic}^2 = 5*E*CVN$$
 (psi sqrt(in), psi, ft-lb) (4)



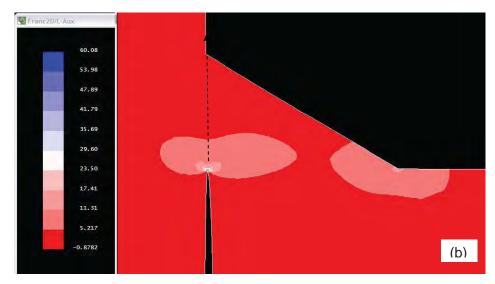


Figure 10. (a) FRANC2D model of cruciform connection tested and analyzed by Kanvinde (2009). (b) Contours of x-stress in weld region at a load, P, of 5kips/inch of weld. *Displacement magnification factor is 100*.

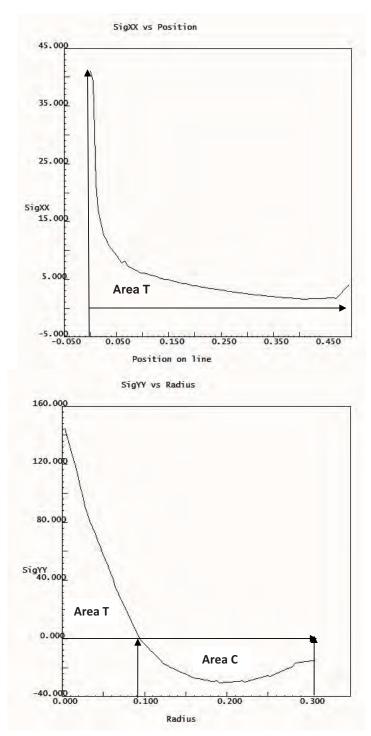


Figure 11. (a) Distribution of x-stress along dashed line shown in Figure 10b, cruciform connection. (b) Distribution of y-stress along line A-B shown in Figure 7a, knife connection.

Equation 4 is used for the conversion from measured CVN values in Table 3, because it has been observed (Tide, 1998; Fisher, 1996) that, at  $21^{\circ}$ C, E70-T4 welds do not produce upper-shelf behavior and toughness. They fail in cleavage at relatively low values of CVN. The theoretical conversion from  $K_{Ic}$  to  $J_{Ic}$  values is:

$$J_{Ic} = K^{2}_{Ic} (1-v^{2})/E$$
 (5)

Note that there was a very wide range of results observed in CVN values for this electrode.

For the upper shelf region of the CVN data,

$$(K_{Ic}/F_y)^2 = 5[CVN/F_y - 0.05]$$
 (ksi sqrt(in), ksi, ft-lb) (6)

Equation 6 is used for the conversion from measured CVN values in Table 4, because it has been observed that, at  $21^{\circ}$ C, the weld materials cited therein produce upper-shelf behavior and toughness. In Table 4, the source data is again CVN values. However, in the Kanvinde study,  $J_{Ic}$  values (in blue) were also obtained through a calibration of finite element models with physical experiments. These values are about four times higher than those expected from the conversion predicted by equation 6. These data show that the post-Northridge toughness-rated electrodes produced  $J_{Ic}$  values 1 to 2 orders of magnitude higher than the lowest value attributed to E70T4 non-toughness-rated electrode. In particular, the  $J_{Ic}$  value used in previous preliminary reports, 0.83 kips/in, is 83 times higher than the lowest value shown in Table 3.



Figure 12. Predicted load-displacement relationship from ANSYS 2D with large displacement theory. The  $J_{IC}$  limit shown is for an upper-bound value of 0.83 kips/in. No shear force included.

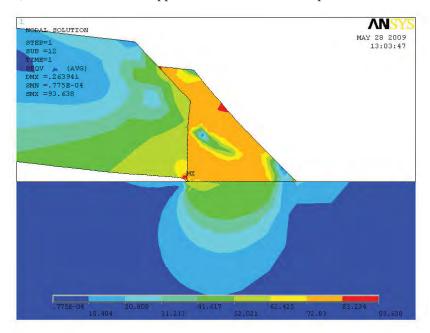


Figure 13. 2D ANSYS-predicted effective stress contours (ksi), on deformed shape (magnification = 1) at a load of 4.7 kips/inch of weld, large deformation theory.

K<sub>Ic</sub> Min<sup>a</sup> **Electrode** CVN Min, J CVN Max, J K<sub>Ic</sub> Max<sup>a</sup> J<sub>Ic</sub> Min<sup>b</sup> J<sub>Ic</sub> Max<sup>b</sup> Source MPa m<sup>1/2</sup> MPa m<sup>1/2</sup> (ft-lb) (ft-lb) kPa m kPa m (ksi in 1/2) (ksi in <sup>1/2</sup>) @21 C @21 C (k/in) (k/in) E70T-4 8 26 31.9 57.1 4.8 15.1 Ojdrovic, 1997 (0.027)(0.086)(6)(19)(29)(52)E70T-4 NA 18.6 NA 49.5 NA 10.9 Civjan, 2000 (avg) (13.7)(45)(0.062)E70T-4 Fisher, 4 29 23.1 2.3 61.5 17 1996 (2.9)(21.3)(21)(56) (0.013)(0.097)

Table 3. Sampled Toughness Information for Pre-Northridge E70T-4 Weld Electrode

# 4.4 ANSYS Elasto-plastic, Large Displacement Analysis on 3D Models of the Knife Connection

Under 1.0D + 1.0SDL + 0.25L, the knife connection analyzed in this report must transmit 41.3 kips of vertical shear, through a bolt group whose centerline is 2.5 inches from the face of the column. Because of this eccentricity, this shear force will increase the tension force, and hence the  $J_I$  value, at the top of the weld, and create a gradient of J-values along the vertical portion of the weld. Also, one-half inch returns were called out on the WTC7 knife connection. Two-dimensional FE models cannot account for effects of this shear force and the returns. Therefore, to compute more realistic values of  $J_I$ , two fully 3D FE models of the connection were used, one without the weld returns, and one with the returns. These models were analyzed using the large displacement option, and with the proper distribution of yield strengths in the column, angles, and weld. In all the 3D ANSYS analyses described herein, quadratic order brick and tetrahedral elements were used, and, except as noted,  $J_I$  values along a crack front were computed directly by ANSYS.

<sup>&</sup>lt;sup>a</sup> using equation 4

<sup>&</sup>lt;sup>b</sup> using equation 5

Electrode	CVN Min, J	CVN Max, J	K <sub>Ic</sub> Min	K <sub>Ic</sub> Max	J <sub>Ic</sub> Min	J <sub>Ic</sub> Max	Source
	(ft-lb)	(ft-lb)	MPa m <sup>1/2</sup>	MPa m <sup>1/2</sup>	kPa m	kPa m	
	@21°C	@21°C	(ksi in ½)	(ksi in ½)	(k/in)	(k/in)	
E70T-7	24.4	25.8	NA	180 <sup>e</sup>	NA	145 <sup>c</sup>	Kanvinde,
8 mm	(17.9)	(19)		(164)		(0.83)	2009
E70T-7-K2	75.9	84	NA	305 <sup>e</sup>	NA	417 <sup>c</sup>	Kanvinde,
8mm	(55.8)	(61.8)		(278)		(2.4)	2009
E71T-8	NA	94.9	NA	111 <sup>d</sup>	NA	56.1 <sup>b</sup>	Civjan, 2000
(avg)		(70)		(101)		(0.32)	

Table 4. Sampled Toughness Information for Post-Northridge Weld Electrodes

#### 4.4.1 Results without Weld Returns

The 3D model without returns has two purposes. First, when analyzed without vertical shear, it provides verification of the J-results from the 2D ANSYS models. Second, when analyzed with vertical shear, it provides a baseline for comparison for the effects of the returns on capacity. Figure 13 shows the ANSYS 3D finite element model excluding the weld returns. Figure 14 shows the results from this model in the form of a plot of maximum  $J_I$  versus applied tension load on the connection; Figure 15 shows a detail of this plot. Figure 14 shows that a tensile capacity of 4.7 kips/inch of weld, previously shown from 2D analysis in Figure 12, is again obtained with an upper-bound value of  $J_{Ic} = 0.83$  kips/inch, when the vertical shear force in not included. This result verifies those of the 2D ANSYS model.

With the shear force acting, Figure 14 shows that the predicted capacity decreases to about 3.6 kips/inch of weld at this upper bound value of toughness. The predicted capacity including shear decreases further when the lower values of  $J_{Ic}$  suggested by Table 3 are used. For example, using the highest toughness value shown in Table 3, 0.10 kips/in, Figure 15 shows that the predicted tensile capacity is about 1.4 kips/inch of weld. Using the lowest value in Table 3, 0.01 kips/in,

<sup>&</sup>lt;sup>c</sup> calibrated using finite element calculations and physical tests

<sup>&</sup>lt;sup>d</sup> using equation 6

<sup>&</sup>lt;sup>e</sup> using calibrated J<sub>Ic</sub> values and equation 5

the weld would have begun to crack at its top under shear alone. Such cracking would have been stable, however, as the J-values decrease when proceeding downwards from the top of the weld and eventually reach zero. However, at this low value of toughness, the connection would have no additional capacity for tensile loading.

## 4.4.2 Results with Weld Returns

One-half inch returns were called out on the WTC7 knife connection. Figure 16a shows a typical view of the deformed shape of the ANSYS 3D finite element model including the weld returns with shear applied. Figure 16b shows a detail of this model around one of the returns.

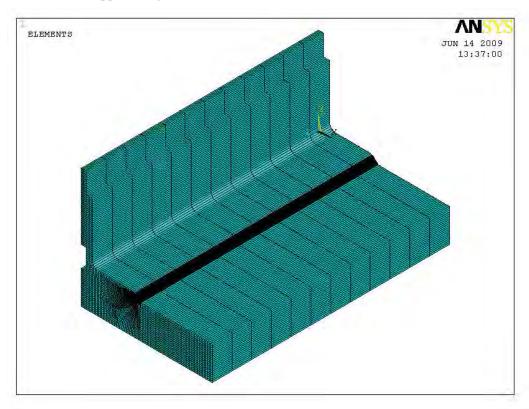


Figure 13. 3D ANSYS FE model without weld returns used to calculate effect of the returns and of vertical shear on  $J_{\rm I}$  values.

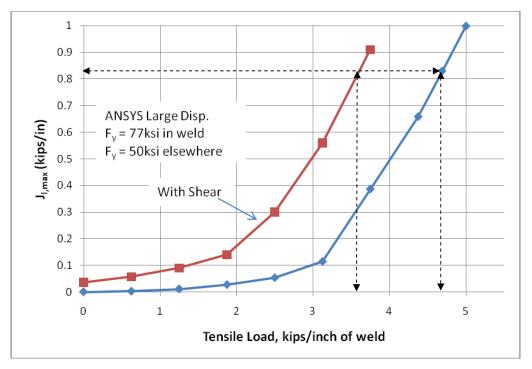


Figure 14. Comparison between maximum  $J_I$  values in the weld, with and without vertical shear force on the connection, no returns. Detail at low values of  $J_{I,max}$  shown in Figure 15.

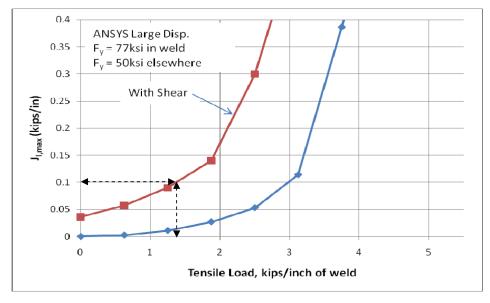
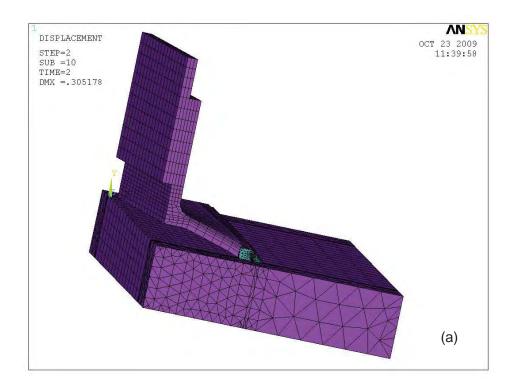


Figure 15. Detail of data shown in Figure 14. Dashed lines indicate capacity at a toughness of 0.10 kips/inch.



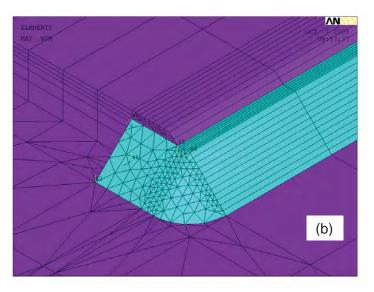


Figure 16. (a) Displaced shape of ANSYS model with weld returns at a tensile load of 3.75 kips/inch of weld, with shear, and a magnification factor of 2, view looking from top to bottom of connection. (b) Detail of model around a weld return.

ANSYS allows sequencing of loads, therefore, in all the results reported here with returns, all of the shear load was first applied, and then the tensile load was applied incrementally. As a check, in an additional analysis both load components were incrementally applied simultaneously. There was less than one percent difference in the maximum J value between the two methods.

Figure 17 shows the predicted relationship between tensile load and maximum value of J along the weld, including shear, with and without the returns. This comparison shows one of the effects of the weld returns: to decrease the  $J_{Imax}$  value for a given tensile load. Figure 18 shows details of this relationship at low values of  $J_{Imax}$ . At a toughness level of about 0.10 kips/inch, the highest measured value shown in Table 3, predicted tensile capacity including shear loading and the returns increases from about 1.35 to about 1.9 kips per inch of weld. At the lowest measured pre-Northridge toughness level of E70-T4 weld material, about 0.013 kips/in, no tensile capacity is available.

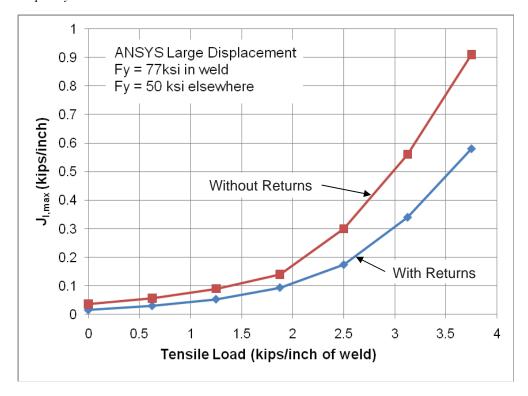


Figure 17. Maximum  $J_I$  along weld versus tensile load, including shear, with and without and weld returns. Detail at low values of  $J_{I,max}$  shown in Figure 18.

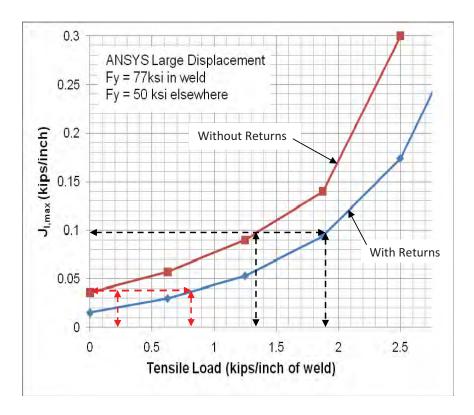


Figure 18. Maximum  $J_I$  along weld versus tensile load, including shear, with and without and weld returns. Detail at low values of  $J_{I,max}$ . Dotted lines indicate capacity at highest measured toughness of pre-Northridge E70-T4 weld material, black, and at median value (0.038 kip/in), red, shown in Table 3.

Also shown in Figure 18 is a predicted capacity of about 0.85 kips per inch of connection depth at a *median* toughness of 0.038 kips/inch. Measured toughness data from the sources listed in Table 3 are not normally distributed, rather they are skewed towards very low values. Therefore, the expected value of toughness is not the mean value.

The second effect is shown in Figure 19. Figure 19a shows the distribution of J values along the entire weld at an example load level. Along the upper return, the only one of concern here, J values decrease to nearly zero at the intersection of the return with the vertical section of the weld, then rapidly rise along this section. Figure 19b compares the J distribution along the

vertical section of the weld, with and without the returns. It can be seen that the return is most effective over only about the first two inches of this section of the weld. In summary, the returns have a significant effect on the maximum value of J along the weld, and change its location from the case without the returns. However, Figures 17 and 18 confirm that, even with returns, the capacity of the WTC7 knife connection predicted using a state-of-the-art analysis is still a small fraction of that predicted using AISC recommended practice.

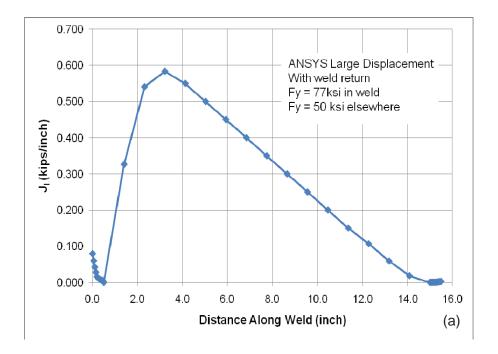
## 5.0 SUMMARY

The purpose of this report was to investigate the capacity of a beam-column knife connection of the type used in the WTC7. This type of connection was designed for transfer of vertical shear load, by longitudinal loading of the fillet welds; however, in supplying lateral restraint against buckling of the column, it would also have to transmit direct tension load, by transverse loading of these welds.

The investigation was confined to analytical and computational activities, comparing capacity predictions based on the state-of-the-practice, according to AISC recommended analysis practice, with those based on state-of-the-art non-linear fracture mechanics calculations via the finite element method.

The salient observations and conclusions of this investigation are:

- The WTC7 knife connection has a large eccentricity in the transverse loading direction.
   No data could be found, from either computation or physical testing, concerning combined longitudinal and transverse capacity of such a design.
- AISC recommended state-of-the-practice predicts a combined longitudinal and transverse
  capacity of the WTC7 knife connection of 27.4 kips per inch of weld, considering weld
  failure only and disregarding flexural failure of the angles.
- This AISC-based prediction is independent of both the degree of eccentricity in the transverse direction and the toughness of the weld material.



- A recent investigation by Kanvinde et al. (2009) involving both physical testing and non-linear fracture mechanics analysis of transversely loaded weld details has shown that capacity of such details is sensitive to toughness, rather than strength, of the weld material. However, this investigation used details with very low transverse eccentricity and its conclusion that AISC practice is applicable to transversely loaded weld details does not cover the WTC7 detail.
- The non-linear fracture mechanics approach presented by Kanvinde *et al.* represents a general, state-of-the-art approach to predicting the combined capacity of a detail with any level of transverse eccentricity under combined loading. This approach was used in this investigation to predict the capacity of the WTC7 knife connection.
- Exploratory 2D finite element implementation of this approach using FRANC2D was first performed. The results of the 2D analyses showed that predicted capacity, under transverse loading alone, was in the range of 3.25 to 3.75 kips/inch of weld, using an

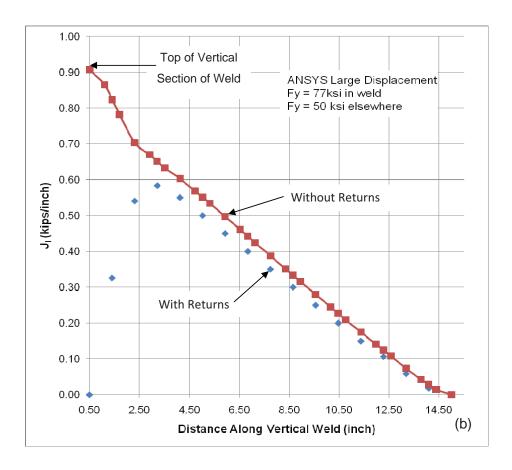


Figure 19. (a) Predicted distribution of J<sub>I</sub> along entire length of weld with weld returns, for tensile load of 3.75 kips/inch of weld. (b) Comparison of distribution of J<sub>I</sub> along length of vertical section of weld with and without weld returns, for tensile load of 3.75 kips/inch of weld.

upper bound toughness for the weld material which is greater than the toughness of the weld material used on the WTC7 connections. Results also showed that large deformation analysis technique would be required because of the significant bending in the outstanding legs of the angles in the WTC7 connection, even under relatively low transverse loading.

• 2D, large deformation analysis was next performed using ANSYS. Using the same upper bound value of toughness which is greater than the toughness of the weld material used

on the WTC7 connections. ANSYS predicted a transverse capacity of 4.7 kips/inch of weld.

- The upper bound value of weld material toughness used in these exploratory studies is characteristic of post-Northridge earthquake welding practice. An investigation of the literature immediately following the 1994 Northridge earthquake event showed that toughness of commonly used pre-Northridge weld materials was significantly lower than this upper bound value. Table 3 shows that limiting values of crack driving force, J<sub>Ic</sub>, for common pre-Northridge electrodes used in beam-column connections are both highly variable and relatively low when compared to those for post-Northridge, toughness-rated electrodes, Table 4. In particular, some measured values of toughness of E70-T4 weld electrode were as low as about 1% of the upper bound value. A measured median value for the pre-Northridge E70T-4 electrode toughness (CVN) data is about 8.25 ft-lb, corresponding to a J<sub>IC</sub> of about 0.038 kips/inch.
- 3D finite element analysis was necessary to capture the effects of shear loading on the detail and of the one-half inch long weld returns called out in the WTC7 connection.
- Two 3D ANSYS models were built and thoroughly analyzed, one with and the other without the returns. Results without the returns under combined loading showed that the predicted connection capacity was about 1.4 kips/inch of weld using the highest measured value of toughness of pre-Northridge E70-T4 weld electrode, Figure 15. Results including the returns showed that the predicted capacity increased to about 1.9 kips/inch of weld, Figure 17. This value is only about 7% of that predicted by AISC recommended practice for weld capacity. At the lowest measured pre-Northridge toughness level of E70-T4 weld material, about 0.013 kips/in, no tensile capacity is available, Figure 18.
- Based on the results from these 3D analyses, and on a review of pre-Northridge weld metal toughnesses, a transverse capacity of the WTC7 knife-type connection of

0.85 kip/inch of weld, based on the median toughness value for pre-Northridge electrodes, is reasonable and recommended.

- The most likely reason for this over-prediction is that the AISC method assumes no bending in the weld. Prying action arising from high eccentricity is not acknowledged: in the limit, the throat area is assumed to be uniformly loaded to F<sub>u</sub> in tension. The 3D nonlinear finite element analyses performed herein show a highly non-uniform distribution of normal stress across the weld throat of the knife connection, including a significant amount of compression, Figure 11b. Although the cruciform configuration used in the Kanvinde studies creates some eccentricity on the weld, it does so only by increasing the thickness of the loaded plate: this results in a much more direct load path through the weld with no prying action and, therefore, a low ratio of bending to normal stress across the weld, Figure 11a.
- Since the capacity predictions using non-linear fracture mechanics depend on finite element analyses, they are subject to modeling errors, and require both verification and validation. Modeling errors can arise from a number of sources, but it is asserted that they are not significant enough here to have any major effect on the principal result: calculated weld capacity is much less than that predicted by the AISC approach. Potential modeling error sources that can be readily identified include:
  - o Idealization of the weld geometry. Some penetration of the fillet weld must have existed in the actual connection. It is also unlikely that the actual weld was uniform in cross-section along its length, or that the returns were exactly ½ inch long. These unknown geometry conditions were neglected here.
  - Idealization of the yield strength distribution in and around the weld. No attempt
    was made to include a HAZ and possible differences in yield strength therein
    compared to base materials and weld material.

Neither of these possible error sources would have a major effect on the highly localized fields around the crack front which are responsible for the corresponding J-value distribution along it. The fact is that there is no load redistribution mechanism available in the WTC7 knife connection. If a tear begins anywhere along the weld, the shed load

must be transferred to another portion of the same, highly loaded weld, causing continued tearing.

Partial verification of the finite element modeling was obtained. The FRANC2D, small displacement results were consistent in expected trend with those obtained using ANSYS 2D, large displacement modeling. These ANSYS 2D results were verified with the 3D ANSYS model without returns and shear loading. Finally, the expected effects of the shear loading and returns were seen in the 3D ANSYS model with returns and shear loading. In all these FE analyses, quadratic order elements were used with tight tolerances on both load and displacement convergence metrics.

## 5.0 REFERENCES

Barsom J M., Rolfe S T, Fracture and Fatigue Control in Structures, Prentice-Hall, 2<sup>nd</sup> Edition, 1987.

Bittencourt T, Wawrzynek P, Sousa J, Ingraffea, AR. Quasi-Automatic Simulation of Crack Propagation for 2D LEFM Problems. *Eng. Fract. Mech.*, 55, 2, 321-334, 1996.

Chi W M, Deierlein G G. Integration of Analytical Investigations on the Fracture Behavior of Welded Moment Resisting Connection, Final Report, SAC Task 5.3.3, Department of Civil and Environmental Engineering, Stanford University, Report #136, 2001.

Civjan S, Engelhardt, M D, and Gross J L, Retrofit Of Pre-Northridge Moment Resisting Connections, *J Struct Eng*, ASCE, **126**, 4, 445-452, 2000.

Fisher J, Dexter R, Kaufmann E J. Fracture mechanics of welded structural steel connections. FEMA No. 288, Report SAC 95-09, 1996.

Kanvinde A M, Fell B V, Gomez I R, Roberts M, Predicting Fracture in Structural Fillet Welds Using Traditional and Micromechanical Fracture Models, *Eng Struct* **30**, 3325-3335, 2008.

WTC7\_Knife Connection\_Report\_Ingraffea

Kanvinde A M, Gomez I R, Roberts M, Fell B V, Grondin G Y, Strength and Ductility of Fillet Welds with Transverse Root Notch, *J Constructional Steel Research*, **65**, 948-958, 2009.

From Kanvinde et al., 2009:

[12] American Institute of Steel Construction, Inc (AISC). Steel construction manual.13th edition. Chicago (IL, USA): American Institute of Steel Construction; 2005.

[17] CSA S16-01. Limit states design of steel structures. Toronto (Canada): Canadian Standards Association; 1989.

[18] American Institute of Steel Construction, Inc (AISC). Manual of steel construction. 2nd edition. Chicago (IL, USA): American Institute of Steel Construction; 1993.

Ojdrovic R P, Zarghamee M S, Fracture of steel moment connections in the Northridge earthquake, *Proc. Instn Civ Engrs Structs & Bldgs*, 1997, **123**, May, 209–217, Paper 11123.

Tide R H R. Stability of weld metal subjected to cyclic and seismic loading, *Eng Structs* **20**, 562-569, 1998.

## 6.0 LIST OF PUBLICATIONS AUTHORED

The following is a list of publications authored by A.R. Ingraffea Ph.D., P.E. in the previous 10 years. (NOTE: *Italics* indicate present or former graduate students or post doctoral associates of Prof. Ingraffea)

## **Book Chapters**

- 1. *Carter B J*, Desroches J, Ingraffea A R, *Wawrzynek P A*. Simulating Fully 3D Hydraulic Fracturing. In **Modeling in Geomechanics**, Ed. Zaman, Booker, and Gioda, Wiley Publishers, pp 525-557, 2000.
- 2. Ingraffea A R, *Wawrzynek P A*. Crack Propagation. In the **Encyclopedia of Materials:** Science and Technology, Elsevier Science, 2001.
- 3. Ingraffea A R, *Wawrzynek P A*. Finite Element Methods for Linear Elastic Fracture Mechanics. Chapter 3.1 in **Comprehensive Structural Integrity**, R. de Borst and H. Mang (eds), Elsevier Science Ltd., Oxford, England, 2003.
- Ingraffea A R. Computational Fracture Mechanics. Volume 2, Chapter 11, Encyclopedia of Computational Mechanics, E. Stein, R. de Borst, T. Hughes (eds.) John Wiley and Sons, 2004, 2<sup>nd</sup> Edition 2008.

#### **Archival Journal Publications**

- 1. Chen C-S, Krause R, Pettit RG, Banks-Sills L, Ingraffea A R. Numerical Assessment of T-stress Computation Using a P-version Finite Element Method. Int. J. Fract., 107:177-199, 2001.
- 2. *Chi W-M*, Deierlein GG, Ingraffea AR. Fracture Toughness Demands in Welded Beam-Column Moment Connections. *J. Structural Division*, ASCE, **126**:88-97, 2000.
- 3. *B Carter, CS Chen*, LP Chew, N Chrisochoides, GR Gao, *G Heber*, AR Ingraffea, *R. Krause*, C Myers, D Nave, K Pingali, P Stodghill, S Vavasis, *PA Wawrzynek*. Parallel FEM Simulation of Crack Propagation -- Challenges, Status, and Perspectives. *Lect. Notes Comput. Sci.*, **1800**:443-449, 2000.
- 4. *Spievak L*, Lewicki D, *Wawrzynek P*, Ingraffea AR. Simulating Fatigue Crack Growth in Spiral Bevel Gears. *Eng. Fract. Mech.*, **68**:53-76, 2001.
- 5. *Pettit R, Chen, C-S, Wawrzynek P*, Ingraffea AR. Process Zone Size Effects on Naturally Curving Cracks. *Eng. Fract. Mech.*, **68:**1181-1205, 2001.

WTC7\_Knife Connection\_Report\_Ingraffea

- 6. *Chen C-S*, Wawrzynek PA, Ingraffea AR. Residual Strength Prediction of Airplane Fuselages Using CTOA Criterion. *AIAA Journal*, **40:**566-575, 2002.
- 7. *Chen C-S, Wawrzynek PA*, Ingraffea AR. Prediction of Residual Strength and Curvilinear Crack Growth in Aircraft Fuselages," *AIAA Journal*, **40**:1644-1652, 2002.
- 8. *Hwang CG*, Wawrzynek, PA, Ingraffea AR. On the virtual crack extension method for calculating the derivatives of energy release rates for a 3D planar crack of arbitrary shape under mode-I loading. *Eng. Fract. Mech.*, **68**:925-947, 2001.
- 9. Lewicki D, *Spievak L, Wawrzynek P*, Ingraffea AR, Handschuh R. Consideration of Moving Tooth Load in Gear Crack Propagation Predictions. *J. Mechanical Design*, **123**:118-124, 2001.
- 10. Cavalcante-Neto JBC, Wawrzynek PA, Carvalho MTM, Ingraffea AR. An algorithm for three-dimensional mesh generation for arbitrary regions with cracks. Eng. With Computers, 17:75-91, 2001.
- 11. *Hanson JH*, Ingraffea, AR. Compression Loading Applied to Round Double Beam Fracture Specimens. I: Application to Materials with Large Characteristic Lengths. *J. Testing and Evaluation*, **30**:508-514, 2002.
- 12. *Hanson JH*, Ingraffea AR. Compression Loading Applied to Round Double Beam Fracture Specimens. II: Derivation of Geometry Factor. *J. Testing and Evaluation*, **30**:515-523, 2002.
- 13. *Hanson JH*, Ingraffea AR. Using Numerical Simulations to Determine the Accuracy of the Size-Effect and Two-Parameter Data Reduction Methods for Fracture Toughness Tests of Concrete. *Eng. Fract. Mech.*, **70**: 1015-1027, 2002.
- 14. *Han T-S, Ural A, Chen C-S*, Zehnder AT, Ingraffea AR, Billington SL. Delamination buckling and propagation analysis of honeycomb panels using a cohesive element approach. *Int. J. Fract.*, **115**:101-123, 2002.
- 15. *Iesulauro E*, Ingraffea AR, Arwade S, *Wawrzynek PA*. Simulation of Grain Boundary Decohesion and Crack Initiation in Aluminum Microstructure Models. Fatigue and Fracture Mechanics: 33rd Volume, In *ASTM STP 1417*, W.G. Reuter and R.S. Piascik, Eds., American Society for Testing and Materials, West Conshohocken, PA, 715-728, 2002.

- 16. *Ural A*, Zehnder A, Ingraffea AR. Fracture mechanics approach to facesheet delamination in honeycomb: measurement of energy release rate of the adhesive bond. *Eng. Fract. Mech.*, **70**:93-103, 2002.
- 17. Riddell WT, Ingraffea AR, Wawrzynek PA. Propagation of non-planar fatigue cracks: experimental observations and numerical simulations. In 33rd National Symposium on Fatigue and Fracture Mechanics; Moran, WY; USA; 25-29 June 2001. pp. 573-597. 2002
- 18. Chew P, Chrisochoides N, Gopalsamy S, *Heber G*, Ingraffea AR, Luke E, *Neto J*, Pingali K, Shih A, Soni B, Stodghill P, Thompson D, Vavasis S, *Wawrzynek P*. Computational science simulations based on web services. *Lect. Notes Comput. Sci.*, **2660**:299-308 2003.
- 19. *Hwang CG*, Ingraffea AR. Shape prediction and stability analysis of Mode-I planar cracks. *Eng. Fract. Mech.*, **71**:1751-1777, 2004.
- 20. *Hanson JH*, *Bittencourt TN*, Ingraffea AR. Three-dimensional influence coefficient method for cohesive crack simulations. *Eng. Fract. Mech.*, **71**:2109-2124, 2004.
- 21. *Ural A, Heber G, Wawrzynek PA*, Ingraffea AR, Lewicki DG, Cavalcante-Neto JB. Three-dimensional, Parallel, Finite Element Simulation of Fatigue Crack Growth in a Spiral Bevel Pinion Gear. *Eng. Fract. Mech.*, **72**:1148-1170, 2005.
- 22. *Hwang CG*, Wawrzynek PA, Ingraffea AR. On the calculation of derivatives of stress intensity factors for multiple cracks, *Eng. Fract. Mech.*, **72**, 1171-1196, 2005.
- 23. Banks-Sills L, Hershkovitz I, *Wawrzynek PA*, Eliasi R, Ingraffea AR. Methods for Calculating Stress Intensity Factors in Anisotropic Materials: Part I z = 0 is a Symmetric Plane, *Eng. Fract. Mech.*, **72**:2328-2358, 2005.
- 24. Cavalcante-Neto JB.; Martha LF, Wawrzynek PA, Ingraffea AR. A Back-tracking procedure for Optimization of simplex meshes, Comm. Numer. Methods Eng., 21:711-722, 2005.
- 25. Banks-Sills L, Hershkovitz I, *Wawrzynek PA*, Eliasi R, Ingraffea AR. Methods for calculating stress intensity factors in anisotropic materials: Part II—Arbitrary geometry, *Eng. Fract. Mech.*, **74**:1293-1307, 2007.
- 26. *Hwang CG*, Ingraffea AR. Virtual crack extension method for calculating the second order derivatives of energy release rates for multiply cracked systems. *Eng. Fract. Mech.*, **74**:1468-1487, 2007.

- 27. *Miranda A, Martha L, Wawrzynek PA*, Ingraffea AR. Surface mesh regeneration considering curvatures, *Eng Comp*, **25**:207-219, 2, 2009
- 28. Coffman V, Sethna J, *Heber G, Liu A*, Ingraffea AR, Bailey N, *Barker E*. A Comparison of Finite Element and Atomistic Modeling of Fracture. *Model Sim Mat Sci Eng.* **16**, 6, 1 September 2008, Article number 065008.
- 29. *Emery J, Hochhalter J, Wawrzynek P,* Ingraffea AR. DDSim: A hierarchical, probabilistic, multiscale damage and durability simulation methodology Part I: methodology and Level I. *Eng. Fract. Mech.*, **76**:1500-1530, 2009.
- 30. *JE Bozek, JD Hochhalter, MG Veilleux, M Liu, G Heber*, SD Sintay, AD Rollett, DJ Littlewood, AM Maniatty, H Weiland, RJ Christ Jr., J Payne, G Welsh, DG Harlow, *PA Wawrzynek*, AR Ingraffea A Geometric Approach to Modeling Microstructurally Small Fatigue Crack Formation- Part I: Probabilistic Simulation of Constituent Particle Cracking in AA 7075-T651. *Model Sim Mat Sci Eng*, **16**, 6, 1 September 2008, Article number 065007.
- 31. *Hochhalter J*, Littlewood D, *Veilleux M*, *Bozek J*, Ingraffea AR, Maniatty A. A geometric approach to modeling microstructurally small fatigue crack formation: II. Simulation and prediction of crack nucleation in AA 7075-T651. *Model Sim Mat Sci Eng*, accepted for publication, 2009.
- 32. Coffman V, Sethna J, Ingraffea AR, Bailey N, *Iesulauro E, Bozek J*. Challenges in Continuum Modeling of Intergranular Fracture. *Strain*, accepted for publication, February, 2010.

# 7.0 LIST OF ALL OTHER CASES DURING THE PREVIOUS 4 YEARS IN WHICH THE WITNESS TESTIFIED AT DEPOSITION OR TRIAL None

## 8.0 COMPENSATION

The hourly compensation rate of A.R. Ingraffea Ph.D., P.E. to date has been \$400/hour.

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**Guy Nordenson and Associates** 

APPENDIX B

Floor Collapse Analysis Report

WTC7 Global Collapse Analysis Report and Summary of Findings – Appendix B 12 February 2010

## APPENDIX B - FLOOR COLLAPSE ANALYSIS REPORT

## WORLD TRADE CENTER 7 COLLAPSE INVESTIGATION New York NY

Prepared for

Gennet, Kallmann, Antin & Robinson PC and Greenbaum, Rowe, Smith & Davis LLP

12 February 2010

By Guy Nordenson PE, SE

Structural Engineers LLP

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## B1.0 INTRODUCTION

This report summarizes the engineering analysis and the findings of Guy Nordenson PE SE regarding the vertical progression of partial floor slab collapse at the northeast corner of WTC7 following the initiating event of Girder 44–79 becoming unseated at Column 79 on Floor 13, one of two collapse initiation theories documented in the report by Dr Colin Bailey (Ref 7).

## B1.1 Description of Progressive Floor Collapse

Upon failure of Girder 44-79's connection to Column 79 on Floor 13, the southern end of the girder is unseated and falls toward Floor 12. As it falls, the composite beams framing into the girder as well as a portion of the concrete slab are also pulled down, and the collapsing partial floor section impacts Floor 12 below.

Using principles of energy conservation, it was determined that the impact energy of Floor 13 falling on Floor 12 is sufficient to fail the floor, causing the propagation of floor collapse on lower floor levels. Using the same methodology, it was determined that the propagation of the floor collapse on lower levels could not be arrested, even on Floors 5 and 7, which are thicker and more highly reinforced than the typical floors. The analysis methods outlined in the following section demonstrate that the failure of the Girder 44–79 connection to Column 79 on Floor 13 initiated a sequence of partial floor collapses that propagated until reaching the base of the structure.

## B1.2 Description of Floor Collapse Analysis Approach

The basis for the analysis was an energy comparison between the remaining potential energy of a floor slab once it has deformed and broken away from its surrounding slab versus the energy required to fail the support structure of the floor below as follows:

A conservative approach to the analysis was taken in which the energy comparison was made on a relative floor-to-floor basis without allowing the potential energy of falling floor areas to accumulate. For example, once it was determined that Floor 12 would fail as a result of the impact of Floor 13, both the remaining potential energy of Floor 13 and its new potential energy due to its mass falling from Floor 12 to 11 were set to zero, and only the new potential energy of Floor 12 falling to Floor 11 was included in the next energy comparison at Floor 11 to assess whether the collapse would propagate further (Figure B1.1).

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While the study was based of necessity upon assumptions about geometry and deformation characteristics of the falling floors, a conservative approach was taken to establish a lower bound potential energy and an upper bound deformation energy, thereby producing the lowest possible shear force transferred to the girder-to-column connection at each level.

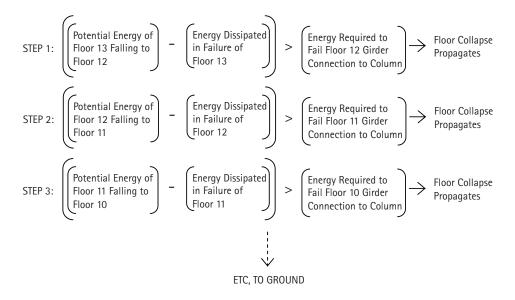


Figure B1.1 Conservative methodology for floor collapse assessment

As the basis for determining both the potential energy of the falling floor and the amount of energy dissipated in its failure, a structural analysis model was generated in SAP2000 Advanced Version 12.0.2 by Computers and Structures Inc of Berkeley CA (Ref 17) to assess the likely geometry of each floor as it collapses due to the failure of Girder 44–79 at Column 79.

The potential energy of each falling floor was calculated as the mass of the deformed floor area tributary to the impact point under the deformed geometry condition multiplied by the height over which that mass would fall before impacting the floor below.

The energy dissipated when a floor falls is the energy required to fracture its continuity with the adjacent structure and to inelastically deform the slab. These energies were calculated as the fracture energy associated with rupture of the concrete and steel in

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the floor slab and the plastic energy from moment-rotation curves for the floor slab. The sources of energy dissipation are as follows (illustrated in Figure B3.1):

- Tensile fracture of highly deformed bays directly to the south and west of the falling floor area
- Shear and tensile fracture of the floor sections in the immediate vicinity of Column
   79
- Plastic hinging of the perimeter of the falling floor area
- Plastic hinging of the borders with the south and west deformed bays (only where trench headers do not eliminate the topping slab continuity)
- Energy dissipation based on the rotational deformation of the falling floor area along hinge lines
- Plastic deformation of falling girder tip at impact with floor below

On floor levels at which the line of slab fracture coincided with the location of a trench header, only the fracture energy of the metal deck and any concrete below the trench header was taken into account.

Subtracting the total dissipated energy from the initial potential energy of the partial floor prior to collapse provided the potential energy of the falling section of Floor 13 at the moment of impact with Floor 12. This potential energy at impact was then converted to an equivalent static force based on the stiffness of the impact location and the resulting girder deflection. The resulting shear force transferred to the connection at Column 79 was then calculated and compared with the expected shear capacity of the connection to determine whether the failure of one floor would cause the failure of the floor below.

This procedure was repeated at each floor level to determine if the partial floor collapse sequence would continue to ground level.

This report details the steps described above as calculated for one typical floor-to-floor stage of the partial floor collapse sequence and includes tables documenting the summary calculations for the full collapse sequence. Additional details of the calculation sequence from Floor 13 to ground can be found in Section B8.0.

## B2.0 FLOOR COLLAPSE ASSUMED GEOMETRY

This section outlines the modeling and analysis methods used to establish the assumed partial floor collapse geometry on which the subsequent calculations are based.

## B2.1 SAP2000 Single Floor Model Analysis

A section of the northeast corner of Level 13 was isolated from the rest of the floor and a single story SAP2000 model of this section was developed. The extents of the partial floor model are shown in Figure B2.1 below. Material properties used in the model were consistent with those noted in Section 3.4 of the main summary report.

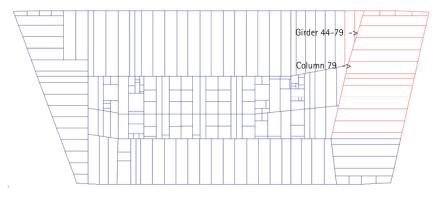


Figure B2.1 Level 13 partial floor SAP2000 model extents (geometry consistent with global model at Level 13)

Gravity loads including dead load, superimposed dead load, and live load were applied to the model assuming a sustained gravity load combination of 1.0DL + 1.0SDL + 0.25LL. The weight of the steel framing was automatically taken into account by SAP2000. The weight of the concrete slab was defined as an area load in accordance with the loading schedule on Sheet S-24 of the structural drawings.

The typical superimposed dead load accounting for ceiling and ductwork, partitions, flooring, and encasement and fireproofing of beams was 25psf based on loading schedule on Sheet S-24 of the structural drawings. Only 25% of the design live load was taken into account, therefore a load factor of 0.25 was assigned to the typical 50psf live load area load case. Superimposed dead loads and live loads for typical floors were assigned according to the loading schedule on Sheet S-24 of the structural drawings.

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The SAP2000 model was intended to provide insight into the locations and modes of element failures preceding floor collapse; it was not intended to recreate the full floor collapse sequence but rather to suggest a probable collapse mechanism that could be accounted for with simple spreadsheets and hand calculations. To simulate connection failure at Column 79 in the model, Girder 44–79 was disconnected from Column 79 and supported by a soft spring.

The resulting slab stresses and deformation characteristics were then studied and utilized to develop a simplified geometrical representation of the partial floor failure that allowed deformation energies to be calculated based on the probable failure pattern suggested by the results of the SAP2000 model.

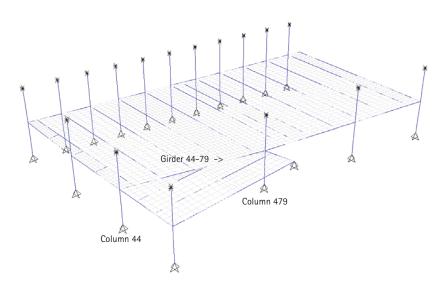


Figure B2.2 Undeformed SAP2000 model with Girder 44–79 connected to Column 79

The idealized deformed geometry of the failed floor slab section was assumed to apply to Floor 13 through Floor 2. Though some intermediate floors such as Floor 6 and Floor 3 had different slab opening configurations, the overall geometry of the isolated floor section was sufficiently similar for the idealized deformation geometry configuration to hold.

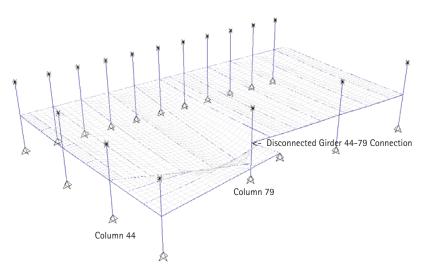


Figure B2.3 Deformed SAP2000 model under gravity loads with disconnected girder (4x amplified elastic deformation)

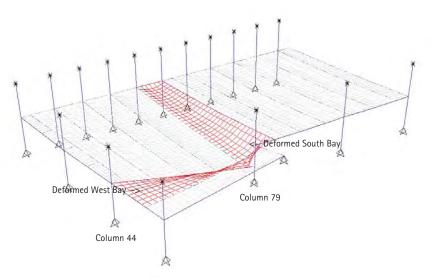


Figure B2.4 Deformed SAP2000 model under gravity loads with disconnected girder (4x amplified elastic deformation) (Note: effect of trench header at south bay not accounted for in approximated deformations)

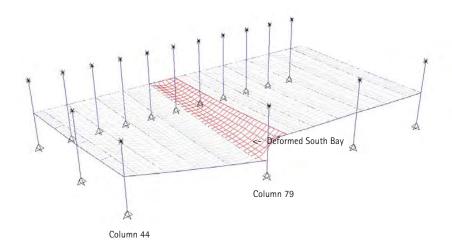


Figure B2.5 SAP2000 Model under gravity loads with disconnected girder (4x amplified elastic deformation; west slab sections removed from view for clarity) (Note: effect of trench header at south bay not accounted for in approximated deformations)

## B2.2 Idealized Collapse Geometry

As shown in Figures B2.3 and B2.4, the unseating of Girder 44–79 from Column 79 caused the south end of the girder to fall, and the girder rotated about its intact connection at Column 44. The composite east–west beams framing into Girder 44–79 were forced to rotate at their connections to the east perimeter framing as they were pulled down with Girder 44–79.

Because the slab on metal deck was connected to the composite beams and girder via shear studs (see Section 3.5.1 of the main summary report), the slab was assumed to take on the faceted deformed shape imposed upon it by the surrounding steel floor framing members.

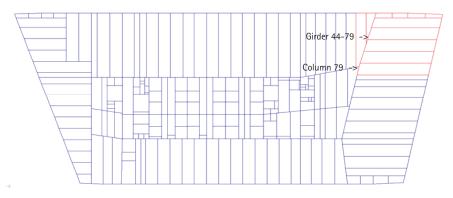


Figure B2.6 Level 13 partial floor extents of idealized geometry model

Based on Figure B2.6 above, the section of Floor 13 affected by the unseating of Girder 44–79 was isolated and the deformation geometry idealized in order to allow for the calculation of deformation values and corresponding energy dissipation at discrete points. The extents of the isolated idealized geometry model are shown in Figure B2.7 below.

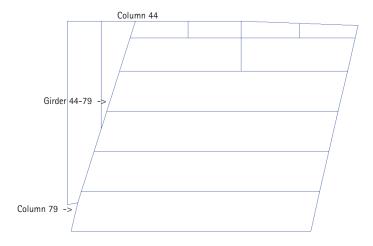


Figure B2.7 Undeformed idealized geometry model extents

Figure B2.8 below depicts the idealized geometry of the Girder 44-79 rotation about its connection to Column 44. In addition to establishing the idealized deformation geometry, this girder rotation diagram served as the basis for determining the girder impact location at the level below. Girder impact is discussed in greater detail in Section B5.1.

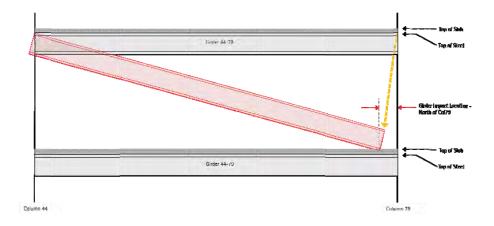


Figure B2.8 Girder 44-79 simplified rotation diagram

Assuming that the beams connected to Girder 44-79 also rotated about their perimeter connections, the idealized deformed geometry model depicted in Figures B2.9 and B2.10 was developed. As these figures illustrate, the idealized deformation geometry accounts for the fact that the free end of Girder 44-79 would have been pulled slightly toward the east perimeter as well as toward the north perimeter as it fell.

For the determination of idealized deformation geometry, framing members were conservatively assumed to rotate about the column centerlines.

As shown in Figures B2.9 and B2.10, the idealized deformation geometry for the collapsed floor section was developed using straight line segments to represent Girder 44-79 as well as the attached east-west composite members. The wind girders on the north and east perimeters were assumed to remain intact and undeformed.

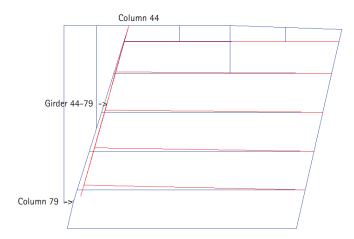


Figure B2.9 Projected plan of idealized deformation geometry (blue represents undeformed geometry, red represents deformed geometry)

At this stage of the analysis, the slab geometry was assumed to remain flat, or undeformed, between each straight line beam segment and then kink at each slab-beam intersection. Subsequent refinements were made to this geometry assumption to more accurately represent and analyze the hyperbolic paraboloid-like shape of the main deformed floor section.

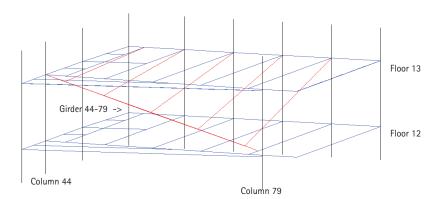


Figure B2.10 Perspective of idealized deformation geometry (blue represents undeformed geometry, red represents deformed geometry)

### B3.0 FLOOR COLLAPSE ENERGY DISSIPATION

Because the energy dissipation is closely related to the idealized deformed geometry described in the preceding sections, the modes and values of energy dissipation will be discussed before the potential energy calculations are described. A number of element deformations were required to take place in order for Girder 44–79 and the attached floor system to collapse and impact the level below in the manner illustrated with the idealized deformed geometry SAP2000 model. Each of these element deformations is a source of potential energy dissipation that must be accounted for in order to determine the reduced energy of the falling floor system at the moment of impact.

## B3.1 Identification of Failure Modes and Energies

Five idealized modes of energy dissipation were analyzed based on the deformed floor system geometry in Figures B2.9 and B2.10. The locations at which these modes of energy dissipation occur are highlighted in the diagram below.

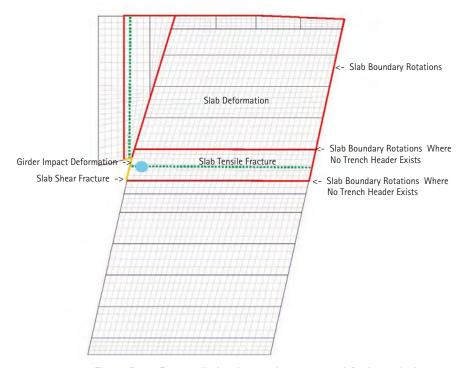


Figure B3.1 Energy dissipation modes accounted for in analysis

The dashed green line represents failure due to tensile fracture across the highly deformed bays of composite slab. The red lines represent slab rotations along boundary lines at which significant angle changes occurred. As most floor levels from Floor 13 to ground had a full-topping-depth trench header along the south perimeter of the isolated slab section, only tensile fracture of the reduced slab section without additional rotational deformations was accounted for at these floors. The orange lines represent shear fracture failure of the short slab segments that connected the highly deformed bays to Column 79. The blue circle represents the plastic deformation of the falling girder upon impact with the floor below.

The fourth source of energy dissipation accounted for in this study is rotational deformation across the main section of collapsed floor along idealized slab hinge lines corresponding to the hyperbolic paraboloid-like shape of the failed floor slab section.

### B3.2 Composite Slab Tensile Fracture

Though some shearing would have occurred as the composite slab highlighted in Figure B3.2 was forced to assume its deformed shape, the deformation shown in Figure B2.4 of the previous section suggests that the primary force the deformed slab bays would have experienced was tension.

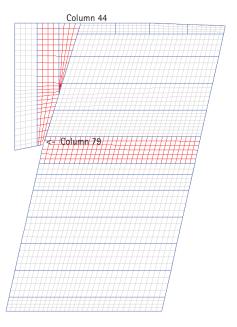


Figure B3.2 "Tensile Fractured" composite slab bays

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As documented in the Table B3.1 below, the composite slab at Floor 13 consisted of 2.5" concrete on 3" metal deck. The profile of the metal deck was based on the standard dimensions provided by the Steel Deck Institute as shown in Figure B3.3. The concrete was reinforced with 6x6 W1.4xW1.4 WWF per the structural construction documents.

Table B3.1 Floor slab properties based upon structural construction documents (top

cover assumed 0.75" when not specified in drawings)

Floor	Direction	Slab Depth	Centroid from Bott	Mom of Inertia	Deck Gauge	Deck Thickness
			[in]	[in4]		[in]
8 - 13	Major	2.5" on 3"	3.2578	107.7	20	0.0359
	Minor	" "	1.25	15.625	20	0.0359
	Angle	" "	use major	use major	20	0.0359
7 (metal deck)	Major	5" on 3"	4.601	348.5	18	0.0474
ueck)	Minor	" "	2.5	125	18	0.0474
	Angle	" "	use major	use major	18	0.0474
7 (8" thick slab)	Both	8" slab	4	512	-	-
6	Major	3" on 3"	3.5347	140.9	20	0.0359
	Minor		1.5	27	20	0.0359
	Angle		use major	use major	20	0.0359
5	Major	11" on 3"	7.6725	2097.5	18	0.0474
	Minor	" "	5.5	1331	18	0.0474
	Angle	" "	use major	use major	18	0.0474
4	Major	3" on 3"	3.5347	140.9	20	0.0359
	Minor	" "	1.5	27	20	0.0359
	Angle	" "	use major	use major	20	0.0359
3	Major	3" on 3"	3.5347	140.9	20	0.0359
	Minor	" "	1.5	27	20	0.0359
	Angle	" "	use major	use major	20	0.0359
2	Major	3" on 3"	3.5347	140.9	20	0.0359
	Minor	" "	1.5	27	20	0.0359
	Angle	" "	use major	use major	20	0.0359

Table B3.1 cont Floor slab properties (assume 0.75" top cover if not specified in dwgs)

Tuoic Bo.1 (	11001 3	iao properties (as.	Sume 0.75 top co	ver ii not specific	ca iii awgs)
Floor	Direction	Top Slab Reinf	Bott Slab Reinf	Added Reinf	Top Cover
		Each Way	Each Way	Perp to Spandrels	[in]
		[in2/ft]	[in2/ft]	[in2/ft]	
8 - 13	Major	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Minor	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Angle	0.040	-	-	0.75
7 (metal		0.04 ("5.040)		0.0 (#4.040)	0.75*
deck)	Major	0.31 (#5@12)	-	0.2 (#4@12)	0.75*
	Minor	0.31 (#5@12)	-	0.2 (#4@12)	0.75*
7 (8"	Angle	0.438	-	-	0.75*
thick slab)	Both	0.72 (#7@10)		-	0.75*
			0.372 (#5@10)		
6	Major	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Minor	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Angle	0.040	-	-	0.75
5	Major	0.6 (#7@12)	-	-	0.75*
	Minor	0.6 (#7@12)	-	-	0.75*
	Angle	0.849	-	-	0.75*
4	Major	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Minor	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Angle	0.040	-	-	0.75
3	Major	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Minor	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Angle	0.040	-	-	0.75
2	Major	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Minor	0.028 (WWF)	-	0.2 (#4@12)	0.75
	Angle	0.040	-	-	0.75

The corrugation of the metal deck was parallel to the tensile fracture across the full extent of both "tensile fractured" bays. As noted above, the location of trench headers along the south edge of the isolated slab section on most floor levels reduced the energy required to fracture the section along these trench header lines due to the discontinuity of the concrete fill and wire mesh reinforcement.

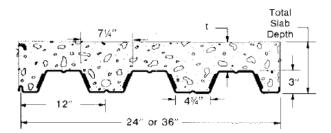


Figure B3.3 Metal deck profile diagram from the Steel Deck Institute

In order to calculate the tensile fracture energy dissipated across the full "tensile fractured" bays, the bays were discretized into a series of 1ft-wide strips spanning from the stationary beam at the south and west edges of the isolated floor section to the deformed beam one bay to the north of the south failed slab section perimeter and to Girder 44-79, respectively. These strips are shown in Figures B3.4 and B3.5.

While the failure of the partial floor slab section may not have fractured along the full lengths of the south and west failed slab section perimeters, it was conservative to assume the fracture occurred across the full length of the perimeters lines in order to account for the maximum possible amount of energy dissipation.

Based on the specific tensile fracture energy values for concrete, metal deck, and wire mesh, the maximum energy dissipation per unit width of slab can be determined for each of the three composite slab components by calculating how much energy is required to fracture each of the composite slab materials. For the purposes of this study, the fracture energy of concrete (Gf) was taken as  $4 \times 10^{-4}$  kip-in/in<sup>2</sup> and the fracture energy of metal deck, wire mesh, and steel reinforcing bars (Gc) was taken as 0.5 kip-in/in<sup>2</sup> (Refs 6 and 10).

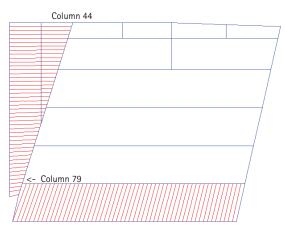


Figure B3.4 Tensile fractured slab section strips plan view

At Floor 13, each 1ft-wide section of composite slab consists of 48in² of concrete, 0.572in² of metal deck, and 0.028 in² of wire mesh. Based on the fracture energies noted above, each strip of slab is therefore capable of dissipating energy equal to:

- $(48in^2 \times 4x10^{-4} kip-in/in^2)$
- + (0.572in<sup>2</sup> x 0.5 kip-in/in<sup>2</sup>)
- +  $(0.028 \text{ in}^2 \times 0.5 \text{ kip-in/in}^2)$
- = 0.3 kip-in

By definition of the specific fracture energies, this cumulative energy in the above calculation represents the full amount required to take the slab section from an unstressed state to failure.

Table B3.2 Shear fracture calculations

Table bo	.z Snea	Hactu	Calcu	10113			140				
Floor	Fracture Length (ft)	Conc Area (in2/ft)	Metal Deck Area (in2/ft)	Wire Mesh or Reinf Area (in2/ft)	Concrete Gf (k- in/in2)	Metal Deck Gc (k- in/in2)	Wire Mesh and Reinf Gc (k- in/in2)	Concrete Energy (kip-in)	Metal Deck Energy (kip- in)	Wire Mesh Energy (kip- in)	Total Energy (kip- in)
13 West	45	48	0.5728	0.028	0.0004	0.5	0.5	0.864	12.9	0.6	14.4
13 South	54	_	0.5728	_	0.0004	0.5	0.5	_	15.5	_	15.5
12 West	45	48	0.5728	0.028	0.0004	0.5	0.5	0.864	12.9	0.6	14.4
12 South	54	-	0.5728	-	0.0004	0.5	0.5	-	15.5	-	15.5
11 West	45	48	0.5728	0.028	0.0004	0.5	0.5	0.864	12.9	0.6	14.4
11 South	54	-	0.5728	-	0.0004	0.5	0.5	_	15.5	-	15.5
10 West	45	48	0.5728	0.028	0.0004	0.5	0.5	0.864	12.9	0.6	14.4
10 South	54	-	0.5728	-	0.0004	0.5	0.5	-	15.5	-	15.5
9 West	45	48	0.5728	0.028	0.0004	0.5	0.5	0.864	12.9	0.6	14.4
9 South	54	-	0.5728	-	0.0004	0.5	0.5	-	15.5	-	15.5
8 West	45	48	0.5728	0.028	0.0004	0.5	0.5	0.864	12.9	0.6	14.4
8 South	54	-	0.5728	-	0.0004	0.5	0.5	-	15.5	-	15.5
7 West	45	48	0.7584	0.31	0.0004	0.5	0.5	0.864	17.1	7.0	24.9
7 South	54	54	0.7584	0.31	0.0004	0.5	0.5	1.1664	20.5	8.4	30.0
6 West	45	78	0.5728	0.028	0.0004	0.5	0.5	1.404	12.9	0.6	14.9
6 South	-	-	-	-	0.0004	0.5	0.5	-	-	-	-
5 West	45	150	0.7584	1.2	0.0004	0.5	0.5	2.7	17.1	27.0	46.8
5 South	54	150	0.7584	1.2	0.0004	0.5	0.5	3.24	20.5	32.4	56.1
4 West	45	54	0.5728	0.028	0.0004	0.5	0.5	0.972	12.9	0.6	14.5
4 South	54	54	0.5728	0.028	0.0004	0.5	0.5	1.1664	15.5	0.8	17.4
3 West	45	54	0.5728	0.028	0.0004	0.5	0.5	0.972	12.9	0.6	14.5
3 South	54	54	0.5728	0.028	0.0004	0.5	0.5	1.1664	15.5	0.8	17.4

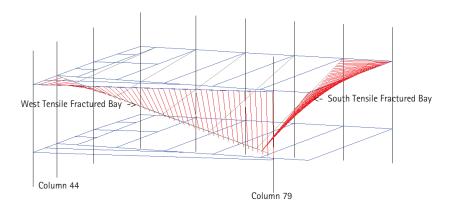


Figure B3.5 Tensile fractured slab section strips perspective view

Based on the method outlined above, the total energy dissipated through the southern bay tensile fracture of Floor 13 was found to be 16 kip-inches. The total energy dissipated through the western bay tensile fracture of Floor 13 was found to be 15 kip-inches. The tensile fracture calculations for Floors 13 through 2 are shown in Table B3.2 above.

### B3.3 Composite Slab Shear Fracture

Principles of fracture mechanics used to calculate the tensile fracture energy of the southern and western slab sections were also used to calculate the energy dissipated due to combined shear and tensile fracture of the short slab segments that link the southern and western bays to Column 79 (See Figure B3.6 below)

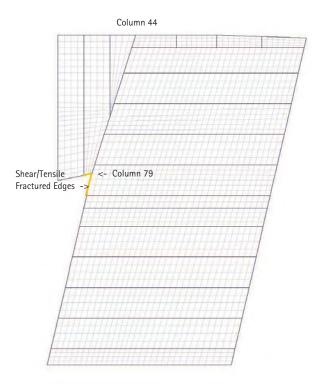


Figure B3.6 Shear/tensile fractured slab boundaries

Both of the boundary edges highlighted in Figure B3.6 were required to fail in order for the floor section to assume the idealized deformed shape. While the concrete portion of the composite slab failed in combined shear/tension mode along these boundary lines, because the specific fracture energy of concrete in shear is significantly greater than in tension it was conservatively assumed that the concrete failed in pure shear fracture mode. The metal deck and slab reinforcing steel were assumed to have failed in tensile fracture mode as they tried to resist the angled pull of the failing floor section.

Assuming maximum slab depth parallel to the flutes of the deck taken over the full length of both boundaries, the cross sectional slab area as well as the area of the metal deck and reinforcing steel were calculated. The material-specific fracture energy values were then applied to these cross sectional areas to determine the energy required to fail the slab boundaries. At Floor 13, fracture of the western boundary of the south bay dissipated 6 kip-inches of energy while fracture of the southern boundary of the west bay dissipated 8 kip-inches of energy, using a constant maximum

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slab depth of 5.5in across the full length. Shear fracture calculations for Floors 13 through 2 are shown in Table B3.3 below.

Table B3.3 Shear fracture calculations

Floor	Max Slab Depth (in)	Mtl Deck Thck (in)	Reinf Area (in2/in)	Shear Length S (in)	Shear Length W (in)	Slab G2c (k- in/in2)	Mtl Deck Gc (k- in/in2)	Reinf Gc (kip- in/in2)	Shear Length S Fracture Energy (k-in)	Shear Length W Fracture Energy (k-in)	Total Shear Fracture Energy (k-in)
13	5	0.0359	0.0023	84	122	0.008	0.5	0.5	5.6	8.1	13.7
12	5	0.0359	0.0023	84	122	0.008	0.5	0.5	5.6	8.1	13.7
11	5	0.0359	0.0023	84	122	0.008	0.5	0.5	5.6	8.1	13.7
10	5	0.0359	0.0023	84	122	0.008	0.5	0.5	5.6	8.1	13.7
9	5	0.0359	0.0023	84	122	0.008	0.5	0.5	5.6	8.1	13.7
8	5	0.0359	0.0023	84	122	0.008	0.5	0.5	5.6	8.1	13.7
7	8	0.0474	0.0258	84	118	0.008	0.5	0.5	9.2	13.0	22.2
6	6	0.0359	0.0023	84	115	0.008	0.5	0.5	6.2	8.5	14.8
5	14	0.0474	0.0500	83	110	0.008	0.5	0.5	14.1	18.7	32.8
4	6	0.0359	0.0023	83	110	0.008	0.5	0.5	6.2	8.2	14.3
3	6	0.0359	0.0023	118	-	0.008	0.5	0.5	8.8	-	8.8
2	6	0.0359	0.0023	-	-	0.008	0.5	0.5	-	-	-

#### **B3.4** Rotational Deformation at Slab Boundaries

In addition to tensile and shear fracture at the southern and western bays of the failed floor section, the idealized deformation geometry also required the slab to bend, or rotate, along boundary lines defined by the straight line deformation geometry. The locations at which the floor section boundaries must rotate in order to assume the deformed shape are highlighted in Figure B3.7 below.

In order to calculate the energy dissipated due to the slab rotations along these boundaries, plastic moment-rotation curves were developed for each boundary slab at each floor level based on the slab properties and orientation. As with the fracture calculations, the slab boundaries were divided into 1 foot segments to allow the hinge properties to be calculated for typical 1 foot widths of slab.

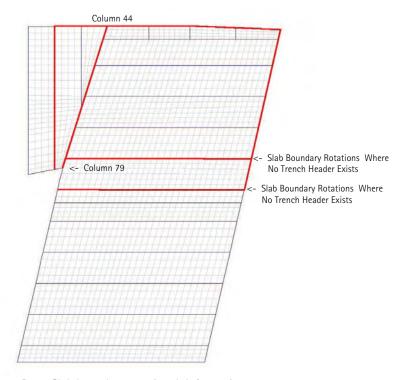


Figure B3.7 Slab boundary rotational deformation

Plastic moment-rotation curves were developed using the material properties for concrete, metal deck, and reinforcing steel noted in Section 3.4 of the main summary report. As the development of moment-rotation curves requires strain compatibility across the section, ultimate strain values of 0.15 for metal deck and reinforcing steel and 0.004 (a conservative value, assuming partial confinement) for concrete were used in this portion of the analysis.

All but two of the slab boundary edges highlighted in Figure B3.7 experienced negative bending (ie top of slab in tension) when the failed slab section took on the deformed shape. The boundary lines along Girder 44–79 and along the beam that frames into the southern end of Girder 44–79 experienced positive bending (ie bottom of slab in tension) when the floor section deformed. As previously noted, the southern boundary energies were typically not included in the energy calculations due to the existence of full-depth trench headers at this location. Where trench headers were not present, the boundary deformation energy was accounted for.

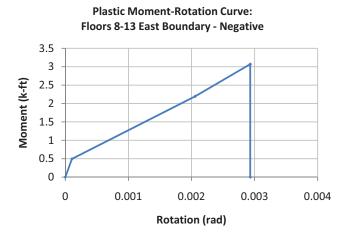


Figure B3.8 Typical composite slab plastic moment-rotation curve

Plastic moment-rotation curves were developed for the appropriate slab bending direction at each of the boundaries (See Figure B3.8 above) by establishing the three points corresponding to concrete cracking, tensile steel yielding, and the governing condition of steel or concrete reaching maximum strain. Plastic moment-rotation curves for slab sections at Levels 8–13 are shown as Figure B3.9 below and detailed for all other floors in Section B8.0. The configuration of the reinforced concrete slabs on composite metal deck used to generate the curves was obtained from the latest structural construction documents.

A common approximation for the plastic hinge length of depth/2 was used. The area under each moment-rotation curve was calculated, representing the maximum energy dissipated by the rotation of a 1 foot-wide strip of slab. The slab boundaries were conservatively assumed to have undergone full plastic rotation, and therefore dissipated the maximum possible amount of energy, along their entire lengths. Additional reinforcing steel in the slab adjacent to the wind girders was taken into account where applicable according to the latest structural construction documents.

Adding these boundary rotation energy dissipation values together at each floor level produced the total slab boundary rotation energy to be subtracted from the potential energy. The values for each floor level are detailed in the tables in Section 8.0.

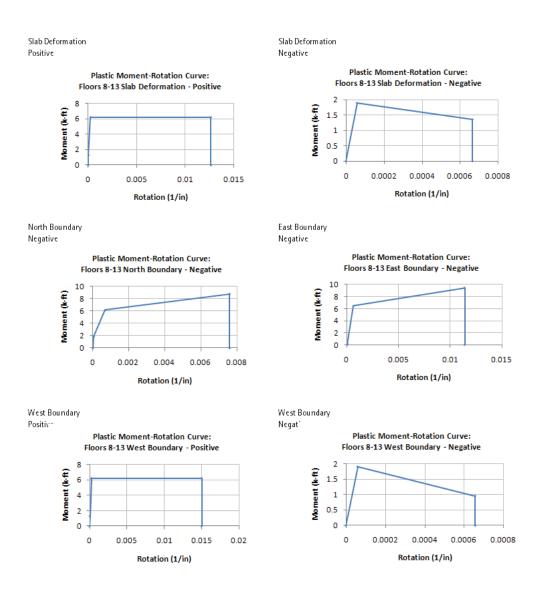


Figure B3.9 Levels 8-13 Composite slab plastic moment-rotation curves

### B3.5 Rotational Deformation at Slab Failure Yield Lines

Using plastic hinge calculation methods identical to those outlined in the previous section, the rotational deformation energy associated with the supported slab section were calculated. As noted in Section B2.2 of this report, the failed floor section distorts into roughly a hyperbolic paraboloid-like shape as it remains supported on the north and east edges while losing support at the south and west edges. This deformed shape is described in Figures B3.10 and B3.11 below.

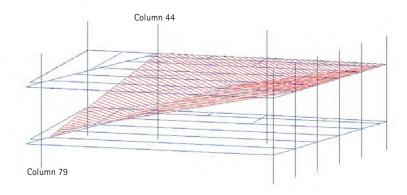


Figure B3.10 Perspective view of floor deformation hinge strips

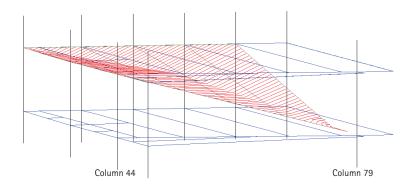


Figure B3.11 Alternate perspective view of floor deformation hinge strips

While it is unlikely that the failed slab section distorted into the exact configuration illustrated in the above figures, the boundary support conditions and the single floor SAP2000 analysis model suggest the geometry as a reasonable approximation for the purposes of this analysis. To determine an upper bound value of the energy dissipated by the failed floor section assuming the hyperbolic paraboloid–like shape, it was assumed that full plastic hinges form along the idealized yield lines depicted in Figure B3.12.

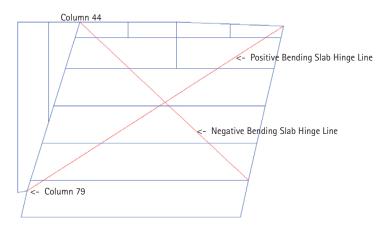


Figure B3.12 Projected plan of idealized floor deformation yield lines

As noted in Figure B3.12 above, two plastic hinge lines, one positive bending and one negative bending, were assumed to form when the failed floor section took on its idealized deformed shape. As in the slab boundary rotation energy calculations, for the sake of conservatism it was assumed that full plastic hinges formed over the full length of both slab hinge lines.

Additional plastic moment-rotation curves were developed for these floor deformation hinge lines in order to account for the additional area of reinforcement in diagonal sections of slab. Detailed moment-rotation curve calculations as well as a table noting the total floor deformation energies for Floor 13 through Floor 2 can be found in Section B8.0.

## B3.6 Falling Girder Plastic Deformation at Impact

The deformation of the falling Girder 44–79 upon impact with the level below is an additional source of energy dissipation. A simplified, conservative approach was used to calculate the plastic deformation energy associated with the local plastic deformation of the portion of the flange that impacts the slab below. The local deformation boundary was taken at a 45 degree angle along half of the bottom flange of Girder 44–79.

This geometry was assumed based on Figure B3.13 below, depicting the angled impact of Girder 44–79 with the floor below, with the full force of impact concentrated at the outer edge of the bottom flange at the south end of the girder.

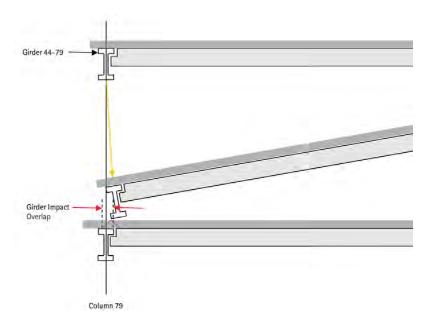


Figure B3.13 Girder impact overlap section diagram (north view)

Based on this impact geometry, which was developed using the simplified deformed geometry previously discussed and the girder impact overlap calculations detailed in the following sections, Figure B3.14 below was developed and used to calculate the energy associated with plastic deformation of a section of flange for Girder 44–79 at all floor levels from Floor 13 to 3.

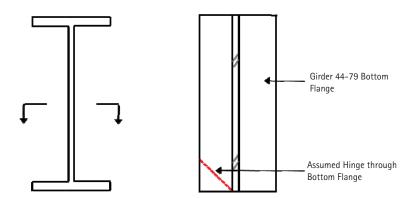


Figure B3.14 Assumed girder plastic hinging at impact (section and plan)

At Floors 8-13 where a partial-depth, full-width bearing stiffener plate was installed at the south end of Girder 44-79, the plastic hinge was assumed to extend through the full height and thickness of one plate. At these floors, the dissipated energy associated with plastic deformation of the girder flange upon impact is 82 kip-inches. The values for all remaining floor levels can be found in Table B6.1 in Section B6.2.

This falling girder plastic deformation at impact energy is the final source of energy dissipation accounted for in this analysis. When combined with the previously described sources of energy loss at each floor level, the total energy dissipation value to be subtracted from the partial floor slab section potential energy was determined.

### B4.0 FLOOR COLLAPSE POTENTIAL ENERGY

The following sections outline the calculation methods used to determine the potential energy of the falling floor section at each level. This potential energy established the energy total from which the deformation energy was subtracted in order to determine the energy remaining when the falling floor impacts the level below.

### B4.1 SAP2000 Single Floor Model Analysis

In order to calculate the potential energy of the tributary floor weight that impacts Floor 12 upon the partial collapse of Floor 13, a partial floor SAP2000 model was used to find the gravity load reaction at the south end of Girder 44–79 where the connection becomes unseated. Column 79 was removed from the model and a roller support was inserted in its place. The gravity load reaction at the former location of Column 79 under the 1.0DL + 1.0SDL + 0.25LL load combination was found to be 46 kips at Floors 13 through Floor 8.

This reaction varies at lower floor levels as the slab configuration, superimposed dead load, and live loads values differ. These different gravity load reactions were calculated at each floor level using additional SAP2000 partial floor models with loading, framing, and slab configurations in accordance with the structural drawings.

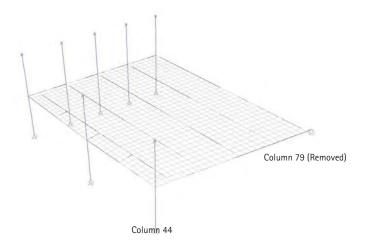


Figure B4.1 Partial floor SAP2000 potential energy model (northwest view)

## B4.2 Equivalent Collapsed Floor Section Geometry

Based on the idealized deformed geometry, it was assumed that the failed floor deformed approximately along the NW-SE slab strip failure lines shown in Figure B3.12. Using these failure lines as a geometrical guide, the floor area in Figure B4.2 was determined to have the equivalent 46 kip total load as the corner reaction found in the SAP2000 model described in Section B4.1.

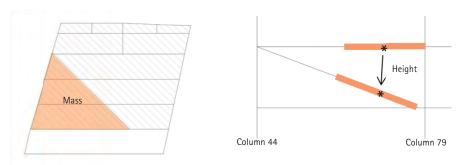


Figure B4.2 Tributary equivalent floor system area along hinge lines

Using the idealized deformed geometry, the centroid of the triangular floor area was traced from its initial position to the height at which it comes to rest when Girder 44–79 rotates and impacts the floor below. The change in height of the tributary equivalent area floor was found to be 83 inches on typical Floors 13 through 8. The calculated fall heights for additional levels are noted in Section B12.0.

In terms of potential energy, the portion of floor not included in the failed floor section was assumed to remain intact around the north and east perimeters of the floor section.

### B4.3 Equivalent Collapsed Floor Section Potential Energy

Based on the weight of the floor system and the distance over which the centroid of the equivalent floor area fell before impact, the potential energy of the slab impact was determined.

Multiplying 46 kips by a height of 83 inches produces an initial potential energy of 3818 kip-inches. This is the balance of energy from which all deformation and failure energies were subtracted at each of the typical floor levels. Potential energy values for all other floors are noted in Table B6.1 in Section B6.2.

### B5.0 FLOOR IMPACT AND FAILURE

The following section outlines the calculations performed to convert the floor system energy at impact to an equivalent force at the face of Column 79. This equivalent static force was then compared to the shear capacity of the girder-to-column connection or connections to determine if the impact energy was sufficient to cause connection, and therefore floor, failure.

### B5.1 Girder Impact Location

Using the idealized deformation geometry illustrated again in Figure B5.1 below, the point at which the falling Girder 44–79 impacted Girder 44–79 at the level below can be determined. Girder 44–79, along with the attached failed floor section, is assumed to have rotated about Girder 44–79's intact connection to Column 44 until the girder impacted the floor below. The rotating girder is assumed to have remained straight on all floors but Floor 13 where thermal deformation due to the fire was approximated. Using the rotated configuration below, the girder impact location was determined.

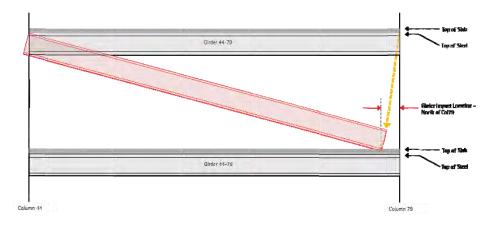


Figure B5.1 Girder 44-79 Simplified Rotation Diagram

As noted in Section B3.2 of this report, the idealized failed floor section geometry takes the eastward rotation of the falling Girder 44–79 into consideration. Using the same geometrical assumptions outlined above it was determined that a portion of the falling Girder 44–79 flange would overlap with a portion of the Girder 44–79 flange below upon impact, as illustrated in Figures B5.2 and B5.3. Therefore, the calculations included no reduction in impact stiffness due to the slight offset. These calculations are detailed in Table B5.1 below.

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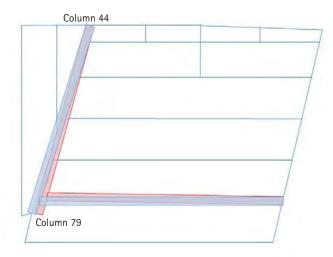


Figure B5.2 Girder 44-79 Impact Overlap Plan Diagram (position of intact girder below show in blue; position of rotated girder at impact shown in red)

Table B5.1 Girder impact overlap geometry

Floor	Fir-to- Fir Height (to below) (in)	Girder 44-79 length (in)	Girder 44-79 depth (in)	Girder 44-79 I (in4)	Girder 44-79 bf (in)	Overall Slab Depth (in)	E-W Beam Length (in)	Girder Impact Location - East (in)	Girder Flange Overlap at Impact Below?	Girder Stiffness at Impact* (k-in)
13	153	547	33.1	6710	11.5	5.5	644	10	YES	-
12	153	547	33.1	6710	11.5	5.5	644	10	YES	139**
11	153	547	33.1	6710	11.5	5.5	644	10	YES	7627
10	153	547	33.1	6710	11.5	5.5	644	10	YES	7627
9	153	547	33.1	6710	11.5	5.5	644	10	YES	7627
8	155.5	547	33.1	6710	11.5	5.5	644	11	YES	7002
7	157	547	30.4	8230	15	8	644	11	YES	8112
6	164	542	27.4	5660	14	6	645	13	YES	3782
5	163	537	27	14668	16	14	647	12	YES	12708
4	168	537	35.9	15000	16.5	6	647	12	YES	11407
3	169.5	537	36.1	16100	16.5	6	647	13	YES	11759
2	135.5	537	35.6	7800	12	6	-	-	-	19044

<sup>\*</sup> Level 12 impact stiffness reduced to based on assumed 80% span length impact point due to pre-failure girder deformation

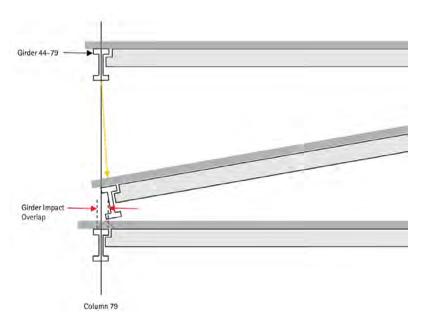


Figure B5.3 Girder impact overlap section diagram

For the impact of Floor 13 on Floor 12 only, the impact location was taken as 1/5 the span length away from the face of Column 79 due to the assumed girder deformation that occurs at Floor 13 due to fire before the girder falls. While the girder deformation does not have a significant effect on the impact location, the assumption that the girder impacts Floor 12's Girder 44–79 at the 1/5 span point is conservative. The assumed geometrical rotation method outlined above is used at all other floor levels.

### B5.2 Girder Stiffness at Impact Location

The stiffness of Girder 44-79 at the location of impact was calculated based on the girder geometry, the girder moment of inertia, and the material properties of steel. The girder stiffness was determined using a simple hand calculation assuming a simply-supported span between column centerlines with a point load applied at the calculated point of impact. This boundary condition assumption was conservative as it accounted for the least stiffness possible.

Taking Young's modulus as 29,000 ksi and using the appropriate girder moment of inertia at each floor level, the girder stiffness K at the point of impact along Girder 44–79 just north of Column 79 was found using the following equation:

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$$K = \frac{3 * E * I * L}{(1 \text{ kip} * a^2 * b^2)}$$

Girder impact locations and corresponding stiffnesses for all levels are outlined in Table B5.1 above.

### B5.3 Impact Force Transmission to Column Connections

Using the girder impact stiffness values, the impact energy was converted to a static force via deformation using the formula:

$$PE = \frac{1}{2} K * D^2$$

where D is girder deflection and K is the girder spring stiffness

This equation can be rearranged and solved for deflection. This deflection value can then be multiplied by the girder stiffness to find the equivalent static force using the equation:

$$F = K * D$$

Taking K equal to 7627 kips/inch yields a static force of 4133 kips. By the geometry established in the girder impact location calculations, the shear distribution of this static force can be determined, allowing the shear force at the face of Column 79 to be calculated.

As shown in Table B5.2, the vertical shear capacity of the seated connection of Girder 44–79 to Column 79 on Floor 13 through Floor 8 was determined to be 632 kips. Expected material properties were considered as defined in AISC/SEI 46–01. No steel strength reduction factors were used in these calculations. The static shear force of 6936 kips as calculated above is far in excess of the connection capacity. Therefore, it is reasonable to conclude that the impact of the partial collapse of Level 13 on Level 12 caused the shear failure of the Girder 44–79 connection.

### B5.4 Connection Failure

Using current AISC-prescribed methodologies and formulas, the expected vertical shear capacity of the Girder 44-79 column connection for the governing failure mode was found at Floors 13 through 2. The typical failure mode was found to be weld shear failure of the seated and knife connections. The typical shear failure value for Floors 13 through 8 was found to be 632 kips.

Expected material strengths listed in Section 3.4 of the main summary report rather than design strengths were used for these calculations in order to give benefit to the structure. Web crippling was checked according to the current edition of AISC and found not to govern the connection failure.

These shear capacity values are noted for all levels in Table B5.2 below. For floors such as Floor 5 at which multiple members frame into the north face of Column 79, all connection shear capacities of members framing into the column were calculated and added together to establish the overall shear capacity at that level. The connection was considered failed after the sum of these capacities was exceeded.

Floor failure was considered to have occurred when the equivalent static shear force as determined using the method specified in the previous sections exceeded the total shear capacity of the girder connection(s) to the north side of Column 79.

Table B5.2 Connection shear capacities

Floor	Conn Type	Vertical Shear Failure Mode	Fexx (ksi)	θ1 (deg)	Lw1 total (in)	Ww1 (in)	θ2 (deg)	Lw2 total (in)	Ww2 (in)	θ3 (deg)	Lw3 total (in)	Ww3 (in)	Rn total (kips)
13	Seated	Weld Failure	77	0	28	0.375	90	8.25	0.375	90	9	0.313	632
12	Seated	Weld Failure	77	0	28	0.375	90	8.25	0.375	90	9	0.313	632
11	Seated	Weld Failure	77	0	28	0.375	90	8.25	0.375	90	9	0.313	632
10	Seated	Weld Failure	77	0	28	0.375	90	8.25	0.375	90	9	0.313	632
9	Seated	Weld Failure	77	0	28	0.375	90	8.25	0.375	90	9	0.313	632
8	Seated	Weld Failure	77	0	28	0.375	90	8.25	0.375	90	9	0.313	632
7	Knife	Weld Failure	77	0	23.5	0.625	0	23.5	0.625	90	1.25	0.313	979
6	Knife	Weld Failure	77	0	17.5	0.625	0	17.5	0.625	90	1.25	0.313	734
5a	Knife	Weld Failure	77	0	21.5	0.313	0	21.5	0.313	90	1.25	0.313	458
5b	Fin	Weld Failure	77	0	18	0.313	-	-	-	-	-	-	184
4	Knife	Weld Failure	77	0	30	0.438	0	30	0.438	90	1.75	0.438	895
3	Knife	Weld Failure	77	0	21	0.438	0	21	0.438	90	1.75	0.438	638
2*	Knife	Weld Failure	77	0	21	0.438	0	21	0.438	90	1.75	0.438	638

#### Notes

Rn = 0.6 x Fexx x (1.0 + 0.5sin^1.5 Theta) x Effective weld area as per AISC Steel Construction Manual Eq J-2

Aw = (sqrt 2 / 2) x Ww x Lw

Lw = Weld length Ww = Weld throat width

 $\theta$  = angle from longitudinal axis of weld Fexx= 77ksi expected for E70 electrodes

<sup>\*</sup> Assumed; available shop drawings depict Column 79 prior to Floor 2 addition in NE corner

### B6.0 VERTICAL COLLAPSE PROPAGATION

The following section outlines the calculations performed to determine the likelihood of the partial floor failure at Floor 13 propagating vertically and causing the subsequent failure of floors below.

### **B6.1** Propagation Analysis Assumptions

As noted in Section B2.2, a conservative approach to the vertical collapse propagation analysis was taken in which the energy comparison was made on a relative floor-to-floor basis without allowing the potential energy of falling floor slabs to accumulate.

For example, once it was determined that Floor 12 would fall as a result of the impact of Floor 13, both the remaining potential energy of Floor 13 and its new potential energy due to its mass falling from Floor 12 to 11 were set to zero, and only the new potential energy of Floor 12 falling to Floor 11 was included in the next energy comparison at Floor 11 to assess whether the collapse propagated farther.

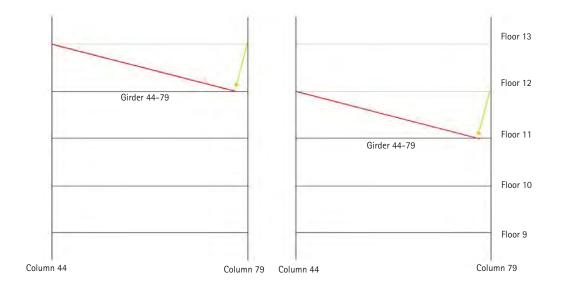


Figure B6.1 Vertical collapse propagation diagram

## B6.2 Floor 13 to Ground Floor Collapse Propagation

As detailed in Section B8, a table of values was developed to compare the equivalent static shear force at the face of Column 79 to the expected connection shear capacity at Floors 13 though 2. When the vertical shear force exceeded the expected vertical shear connection capacity, the partial floor slab section tributary to Girder 44-79 was considered to have failed, and the floor collapse propagated to the next level.

As noted in Sections B2.2 and B6.1 of this report, the accumulation of additional mass that occurred as the partial floor collapse sequence progressed lower was not taken into account in order to maintain a lower bound value of potential energy throughout the analysis, thereby adding an additional degree of conservatism. Thus, the impact energy at each floor level was based solely on the potential energy of the falling partial floor section from one level directly above.

As illustrated in Table B6.1 below, the shear capacities of the Girder 44–79 connections at Column 79 were insufficient to arrest the collapse sequence on all levels from Floor 13 to the ground. Beginning with the impact of Floor 13's Girder 44–79 on Floor 12, the collapsing floor slab section caused the connection failure of Girder 44–79 at the level below. In this way, the partial floor collapse sequence propagated from Floor 13 to the ground.

Table B6.1 Summary table for floor collapse calculations illustrating floor failure at all levels from Floor 13 to the ground

	ENERGY TOSS							IMPACI ENERGY	INTAL HORE AND PREDICT	ANDINAMINA				
Floor Level Potential Energy (Sun)	South Bay Tensile Fracture Energy (N-m) <sup>a</sup>	West Bay Tensile Fracture Erergy (R-in) <sup>III</sup>	Slab Boundary Rotations (fr-m)**	Slab Deformation (K-in)*	Slab Shear Re-	Grider Impact Corner Yielding (Runiff)	Tota Dissipated Energy (K-in) ****	Remaining Energy at Impact (K-in) <sup>9</sup> .	Girder 44-79 Stilfness Below at Impact (K/in) x	Equiv Static Force at Impact (Aps)	Shear at Pace of Col 79 (kips)***		Shear Capacity of Col 79 Conn Below (kips) <sup>34</sup>	Connection Street
	- 41	14	147	70	14	83	346	3473	188	984	787	3A	632	1
	17	14	147.	70.	14	82	345	3473	7627	7279	6915	57	532	FAILVIRE
	- 11	14	147	7.0	14	82	345	3473	7627	7279	6915	30	632	FAILURE
	21	5.4	147	- 0.2	- 11	82	345	3473	7537	7279	3169	19	632	FAILURE
	17	3.4	147	70	14	82	345	3473	7002	6975	9299	25	632	FAILURE
	21	.14:	147	30	14	82	345.	3838	8112	2573	7194	8/	632	FAILURE
	31	25	518	302	.23	09	656	4210	3782	5643	1985	\$A	626	FAILURE
	-	- 51	091	105	91,	83	348	4012	12708	10038	2656	\$A	734	FAILURE
	35	.25	2238	1071	33	V6V-	55.55	4467	11407	(0095	1.656	SA	1190	TANCHE
	21	15.	244	- 105	15	- 16	488	7987	11759	13706	13020	7.7	388	FAILURE
	17		168	-111	6	301	410	7217	19044	16579	05251	3/	838	EAILURE .
	-		,	1										FAILURE

Calculated based on floor slab gravity reaction at Co179 and reduced floor-to-floor height of equivalent failed slab section centroid

Calculated using tensile fracture energies of GF = 0,0004 k-in/in2 for concrete and Gc = 0,5 k-in/in2 for metal deck, wire mesh, and rebar for 1ft-wide slab sections

See note ii above

Calculated using area under stab section plastic moment-intation curves and assuming full plastic ninge formation along entire length of all boundaries

Calculated using area under slab section plastic moment-notation curves and assuming full plastic thinge formation along entire length of all idealized slab hinge lines

Concrete fracture mode assumed to be pure shear across entire slab edge length

Sum of South Bay Tensule Fracture, West Bay Tensile Fracture, Stab Boundary Rotations. Stab Deformation, and Stab Shear Calculated assuming formation of full plastic hinge along corner of Girder 44-79 upon impact with floor below

Remaining Energy at Impact = Floor Level Potential Energy, -Total Dissipated Energy

Girder stiffness at level below failed state calculated based on assumption of striply supported beam using K = 31E\*\*\*L / H\* a 216\*21 where L = 29000ks

Force = K \* Deflection(D) where D = SQRT(2\*Energy/K) based on PE= 0.5\*K\*D\*2

By geometry of impact location

Determined through series of AISC-prescribed calculations using expected material strengths

If shear at face of column > shear capacity of connection below, shear failure assumed to occur

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### B7.0 CONCLUSIONS

Through the use of simplified, conservative hand calculations and basic principles of physics and mechanics, this report demonstrates that the unseating of Girder 44–79 at Floors 13, or at a lower floor in the building, initiates a sequence of partial floor collapses all the way to the ground. The analysis approach undertaken is transparent and straightforward and does not rely on a complex computer analysis that may obscure assumptions inherent to the process. A reasonable lower-bound potential energy was calculated at each floor level. From this minimum initial energy, an upper-bound floor failure and deformation energy was subtracted, thereby yielding a conservative impact energy use to assess the failure at each floor level.

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## B8.0 MOMENT-ROTATION CURVES AND HINGE ENERGIES

The tables on the following pages document the development of the plastic moment-rotation curves used to determine the slab boundary and slab deformation energy dissipation values at Floor 13 through Floor 3.

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HOOR SLAB COLLAPSE ANALYSIS MOMENT-ROTATION CURVE SUMMARY

Page B41

Table B8.1 Moment Rotation Curve Summary

**Guy Nordenson and Associates** 

WTC7 Global Collapse Analysis – Appendix B Floor Collapse Analysis Summary 12 February 2010

	Wed Slab Boundary	Positive Bending	-		12	Znut - Roundary (k-in)	total - Level 2 (k-m)
		Wegstive Sending	0.188	0.0	8.0		
	Arepunog gels unos	Positive Bending	0,180	tr'i.	0.08		
	Viebnuod del 2 ized	Megative Bending	0'609	20	67.		
	North Slab Boundary	Megative Bending	0.269	13	191		
		Megative Bending	93678	0.0	6'0	108	
7,400	Slab Deformation	Positive Bending	9130	6.1	1,701	Sum - Deformation (k-in)	
	E1 - 1	Megative Bending	-	-	-	191	278
	Viebnuod del2 \$29V4	Positive Bending	-			Sum - Boundary (k-m)	Total - Level T. (k-in)
		Megative Bending	700,0	0.0	7.0		
	South Slab Boundary	Positive Bending	7000	4.1	82.2		
	Viebriuod del 2 ised	Megative Bending	9,609	20	6.7	1	
	North Slab Boundary	Megative Bending	0,968	- 61	77.1		
		Megative Bending	770.0	0.0	8.0	111	4
£400	Slab Deformation	Positive Bending	935.0	1.1	L'60T	(ni-a) nottemtotati - muz	
	7	Megative Bending	2380	0.0	9'0	511	320
	West Slab Boundary	Positive Bending	937.6	1.1	63.1	Sum - Boundary (k-in)	Total - Level - Tetol
		Megative Bending	0.858	0.0	8,0		
	Yashruod dal2 itino2	Prisitive Bending	07/49	1.4	76.0		
	East Slab Boundary	Negative Bending	D'609	0.2	61		
	Victoria Slab Boundary	Negative Bending	870.0	13	F98		
		Negative Bending	730C	0.0	8.0	501	
6 100	Slab Deformation	Posttive Bending	991°C	1.1	970L	Sum Deformation (k in)	
	TO STATE FOR	Megative Bending	2330	5.4	2023	2238	3303
	West Stab Boundary	Positive Bending	2380	EII	9,702	Sum - Boundary (k-in)	Total - Level 5 (te-in)
	The Color was	Megative Bending	0.4848	St	243.0		
	Ynabruod dele diuoe	Positive Bending	0.47.0	140	2,867		
	East Slab Boundary	Megative Bending	0,600	irt .	242.3		
	Vistandary Slab Boundary	Megative Bending	0,698	4.6	1,445		
		Megative Bending	730,0	6°E	234.8	1701	
5,400	Slab Deformation	Positive Bending	0,168	EIL	Z'9ER	(m-A) nothermoted - mu2	
					C0		
	CANDON STORY	Megative Bending	652	0.0	2.0	091	265
	West Slab Boundary	Positive Bending	E+5	1/1	63,8	(mi-xl) ynsbnuod - mu2	Total - Level 6 (k-in)
	A ST THE STORY	Megative Bending	-		-		
	South Slab Boundary	Pusitive Bending		-:-	-		
	Yashinud del2 tes3	Megative Bending	8,808	2.0	6.7		
	Yrishmod del2 dinuM	Megative Bending	5.187	6.1	8.78		
		Megative Bending	713	0.0	8,0	102	
9 100	Slab Deformation	Positive Bending	688	4.1	1043	Sum - Deformation (k-in)	Ī
	7. 7	Megative Bending	979	6.1	1/69	ULG	620
	West 5lab Boundary	Positive Bending	248	3.2	Z5+t	Sum - Boundary (k-in)	Total - Level 7 (K-in)
		Negative Bending	647.8	4.1	73.3		
	Airpanog qets innos	Positive Bending	5.7.43	3.2	9,171		
	East Slab Boundary	Negative Bending	7,800	9'0	560		
	Vision Stab Boundary	Megative Bending	8.881	9.0	42.6		
		Megalive Bending	007	5.1	939	205	
L 100	Slab Deformation	Positive Bending	888	3.2	E,EES.	nortemoral - muz	
	Action and the	Megative Bending	230	0.0	5.0	141	717
	West Slab Boundary	Positive Bending	149	1.1	7,03	(ut-a) Ampunog - ums	Total Levels 8-13 (K-In
		Megative Bending	-	-	- 8		
	Arepunog gels upnos	Positive Bending			5	-	
	East Slab Boundary	Megative Bending	7,808	1.1	9'5'5	1	
	Viennog del Annon	Иедайче Велана	7852	1.0	LZh	-	
	The state of the s	Negative Bending	007	0.0	10	Ď/	
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CL-9 Sign	Slab Deformation	Positive Bending	888	60	1,63	Sum Deformation (k-in)	

C7 Global Collapse Analysis or Collapse Analysis Summar

FLOOR SLAB COLLAPSE ANALYSIS PLASTIC HINGE ENERGIES

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APPENDIX C

Interior Column Stability Analysis Report

WTC7 Global Collapse Analysis Report and Summary of Findings – Appendix C 12 February 2010

APPENDIX C - INTERIOR COLUMN STABILITY ANALYSIS REPORT

# WORLD TRADE CENTER 7 COLLAPSE INVESTIGATION New York NY

Prepared for

Gennet, Kallmann, Antin & Robinson PC

Greenbaum, Rowe, Smith & Davis LLP

12 February 2010

Βv

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WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

# C1.0 INTRODUCTION

This appendix provides additional information on the methodology and results of the column stability analyses conducted on the eastern interior columns of WTC7. The purpose of the analyses was to assess the stability of Columns 79, 80 and 81 in World Trade Center 7 (WTC7) following the initiation of collapse (both Scenarios A and B identified in the main summary report) based on the strength and stiffness of their lateral bracing conditions.

# C2.0 BACKGROUND ON THE ASSESSMENT OF COLUMN STABILITY

In order to provide context to the column stability analyses that were carried out for the eastern interior columns of WTC7, this section reviews the fundamentals of column stability theory, highlights the vulnerabilities associated with the lateral bracing of the WTC7 interior columns, and provides an explanation for the analysis method that was used.

# C2.1 Stiffness and Strength Requirements for Adequate Lateral Bracing

An ideally straight column does not impose any lateral loads on its bracing members until it reaches its critical load capacity (or buckling load,  $P_{\rm cr}$ ). If its bracing member is a sufficiently stiff spring, the column will maintain its position at the brace and buckle in two half waves above and below the bracing location at four times the load (4\* $P_{\rm cr}$ ). However, if the bracing member is a relatively flexible spring, it will not be sufficient to prevent the column from buckling in a single wave, which will occur at a load only somewhat higher than the buckling load,  $P_{\rm cr}$  (Figure C2.1a–d) (Ref 15).

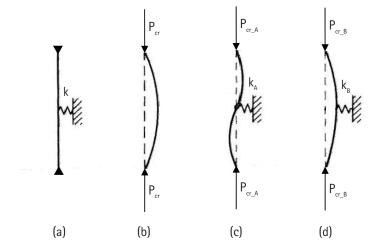


Figure C2.1a–d Effect of brace stiffness on the buckling of an ideal column. (a) unloaded column with lateral brace (b) buckled column without intermediate brace (c) buckled shape for stiff spring as intermediate support (d) buckled shape for flexible spring as intermediate support.  $P_{cr\ A} > P_{cr\ B}$  (Ref 15)

In reality, columns are not ideal and they have imperfections, including initial out-of-straightness due to allowable fabrication and erection tolerances, which impart lateral forces on floor structure when the columns are loaded vertically (Figure C2.2). When a column has adequately stiff and strong lateral bracing, the secondary effects of the imperfections are negligible. However, if a column is not sufficiently braced, the effect of the crookedness may be amplified, leading to buckling (Ref 15).

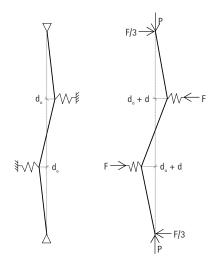


Figure C2.2 Lateral forces and displacements resulting from initial crookedness of column

Section 5.31 of William McGuire's *Steel Structures* (Ref 15), first published in 1968, provides a methodology for determining for simple cases the stiffness and strength required to adequately brace a column using simple hand calculations. The required stiffness of a lateral brace can be determined directly from the ideal case of a straight, axially-loaded column (Figure C2.3). The required strength, however, must be established based on the assumption of imperfections in either the geometry of the member (ie crookedness) or the loading (ie non-axial forces). Typically the crookedness assumed on a column is correlated to its expected buckling shape.

Depending upon the assumptions made about imperfections and the number of braced stories, McGuire calculates that the required force to adequately brace a column ranges approximately between 0.7% and 3% of the buckling load of the column ( $P_{cr}$ ). He also states that the calculations "support a frequently used rule of thumb that bracing having a capacity on the order of 2 percent of that of the main member will provide full support." (Ref 15)

The governing edition of the Building Code of the City of New York (Ref 8) at the time of WTC7's design contained a provision for the required axial capacity of members providing bracing to columns that was consistent with this statement. The excerpt from the Building Code of the City of New York is shown in Figure C2.4. In this standard, the 2% bracing requirement is a function of the axial load in the column rather than its buckling load, so the magnitude of the 2% cannot be directly compared to the percentages calculated by McGuire. The 2% requirement applies to the sum of the capacities of the members bracing a column in each direction, major or minor.

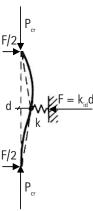


Figure C2.3 Calculation of required stiffness ( $k_{id}=2P_{cr}/I$ ) for simple ideal case using small deflection theory (Ref 15)

C26-1001.2 Bracing. — Unless otherwise specified in the reference standards, members used to brace compression members shall be proportioned to resist an axial load of at least 2 per cent of the total compressive design stress in the member braced, plus any transverse shear therein.

Figure C2.4 NYC Building Code excerpt regarding lateral bracing (Ref 8)

### C2.2 Actual Conditions of WTC7 Columns

### C2.3.1 Out-of-Straightness Conditions

In order to perform a stability analysis, an initial out-of-straightness must be applied to the column. AISC design column bracing specifications use a slope of 1:500 to establish minimum brace forces. At the time that WTC7 was constructed, the maximum allowable erection tolerance according to the AISC Code of Standard Practice for deviation from a plumb line was 1:500, and the working points of splice levels could not fall outside a horizontal envelope of 1.5" from the plumb line (Ref 2 and Figure C7.7 Ref 3).

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

In reality, the out-of-straightness of the eastern interior columns may have been greater than its initial erection tolerances as a result of building movements and deformations induced by the northeast floor collapse described in Stage 1 of the collapse sequence (see Appendix B of the main summary report) and the thermal effects on the floor framing of fire on numerous levels of the building. Therefore, the crookedness of 1:500 used in the stability analyses, without consideration for additional possible deformations due to the fires and floor failures, is considered to be a reasonable, if not conservative, estimate of the actual crookedness of the columns immediately before buckling.

The girders and floor beams framing into and providing lateral bracing to the interior columns were therefore subjected to lateral loads as a result of this out-of-straightness due to the gravity loads in the columns.

## C2.3.2 Bracing Conditions

The majority of the girders and floor beams framing into the interior columns of WTC7 were connected to the columns by welded double-angle knife connections. The capacity of each connection governed the overall capacity of the bracing member. As documented in the report by Dr Anthony Ingraffea (Appendix A of the main summary report), these knife connections were weak in tension due to their susceptibility to weld fracture. Because many of these double-angle knife connections were used in cases where a column was braced on only one side, they were required to work in both tension and compression to brace the column. Therefore, the weakness of these connections in tension governed their ability to provide bracing to the columns. As explained in Section 4.2.3 of the main summary report, the concrete floor slabs of WTC7 were unable to contribute to the capacity of the lateral bracing system of each column.

Tables 4.2 through 4.25 in the main summary report provide a comparison between the code-prescribed 2% bracing requirements for the WTC7 interior columns and the design capacity of the girder and floor beam connections that braced these columns. Design capacities rather than expected capacities are presented in these tables to illustrate the code check that the Engineer of Record should have made during the design process¹. The tables highlight that over 46% of all floor-to-interior column joints in the building did not meet the 2% code requirement in at least one direction.

The tables in Sections C.5 and C.6 provide additional information on the interior column connections and the bracing capacity calculations summarized in Tables 4.2 through 4.25 of the main summary report.

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

<sup>&</sup>lt;sup>1</sup> In reality, according to Dr Ingraffea's fracture analysis of double-angle the knife connections the actual bracing provided to many columns was even lower than these values.

## C2.3.3 Buckling Vulnerability

WTC7's conditions of allowable out-of-straightness and bracing weakness described above indicate that the interior columns were vulnerable to buckling. As described in the next section, a methodology was therefore developed to assess whether under these allowable out-of-straightness parameters, the weak double-angle knife connections had sufficient stiffness and strength to provide adequate bracing to the eastern interior columns and to allow them to carry service loads as adjacent floor structure was lost.

## C2.3 Computer Analysis Methods to Assess Column Stability

The methodology described in McGuire for determining the required stiffness and strength of a member to adequately brace a column is relatively straightforward for a column with a limited number spans; however, the analysis increases in complexity for a column with up to 48 spans and varying section properties and axial loads along its height, such as is the case for each interior column of WTC7. For this reason, a methodology for assessing column stability using a computer structural analysis program was sought.

In general there are two types of computer analyses used to assess the stability of a column. The first is a linear buckling analysis, an eigenvalue analysis that considers an ideally straight column and determines the modes of instability of the column due to a specific loading condition. The results of this type of analysis are consistent with Euler buckling formulas. The second type of stability analysis is a second-order geometrically nonlinear analysis, which considers a column with either an initial crookedness or an imposed lateral load in addition to a vertical load.

A linear buckling analysis is often used to calculate the buckling factor for a column with fixed support conditions and no intermediate supports, such as a pin-ended, single-span column. Also, because the analysis is able to take into account the effect of the stiffness of lateral bracing, it can be used to accurately determine the buckling behavior of a column with intermediate supports defined as springs. However, because a linear buckling analysis deals with ideally straight members, it is not possible to use this type of analysis to determine the resultant force on a brace due to a specific buckled form or the required capacity of the brace to activate a certain buckling mode.

A second-order stability analysis, however, is able to account for the effects of both the stiffness and the strength of intermediate lateral bracing on a column. It can also take into account material nonlinearities such the presence of a finite lateral brace capacity. Furthermore, it is able to analyze the effect of different initial crookedness configurations. As a result, it was determined that a second-order nonlinear analysis

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was the most applicable approach for assessing the effect of the weak lateral bracing conditions of WTC7's interior columns and the susceptibility of these columns to buckling.

The analyses were performed using SAP2000 Advanced Version 12.0.2, a structural finite element analysis program developed by Computers and Structures Inc of Berkeley CA. Because of the complex interactions of the nonlinear lateral supports and the tendency for numerical instabilities resulting from the oftentimes simultaneous failure of these lateral supports, a dynamic time-history (direct integration-type) second-order analysis was used instead of a static one. This approach provided better stability, and the loads were applied quasi-statically with 99% damping in order to minimize the dynamic effects. The nonlinear features of the analysis included both geometric nonlinearity (P-Delta plus large displacements) and nonlinearity of the lateral bracing capacities. No other material nonlinearities were considered in the analysis – the material behavior of the column itself was elastic.

Prior to running each second-order analysis, a linear buckling analysis was performed to assess the most probable buckling shape, which then informed the initial crookedness used in the second-order analysis. Refer to Section C3.2.5 for additional information.

## C3.0 INTERIOR COLUMN STABILITY ANALYSES

This section provides details of the stability analyses conducted for each of the eastern interior columns in WTC7.

# C3.1 Basis of Buckling Sequence

The bracing conditions used in the stability analyses for Columns 79, 80 and 81 were based upon an assumed sequence of failure corresponding to the probable global collapse sequence detailed in Section 5.3 of the main summary report.

Figure 5.3 in the main summary report illustrates that Column 79 is first vulnerable to buckling following the failure of the northeast floor areas on the lower levels of the building due to the weakness of the remaining double angle knife connections in the south and west directions. Immediately following the failure of Column 79 and the loss of the floor structure to the north and east of Column 80, Column 80 becomes susceptible to buckling. After Column 80 buckles and the floor areas supported by the two transfer trusses collapse, Column 81 loses its western brace and becomes vulnerable to instabilities along its minor axis. The analyses described in Section C3.2 were conducted in the order described above to validate this sequence of column buckling and show that each vulnerable column would have buckled under the loads it was carrying at the time of collapse.

The other twenty-one interior columns west of Columns 79, 80 and 81 were not analyzed for stability because other mechanisms are responsible for their failure as described in the probable collapse sequence in Section 5.3 of the main summary report. Columns 76, 77 and 78 collapse due to the failure of Transfer Trusses 1 and 2. The remaining interior columns to the west then fail due to the rupture and instability of the floor diaphragm. Based on their pervasive lateral bracing code violations and the prevalence of the fracture-susceptible double-angle knife connections used to brace them, it is probable that these other interior columns would have buckled sequentially as their adjacent floor areas failed had other mechanisms not caused them to fail.

## C3.2 Interior Column Stability Analysis Input

This section provides a basis for the assumptions used in the stability analyses for the three eastern WTC7 interior columns. Additional documentation of the analysis assumptions is provided in Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report.

# C3.2.1 Loading

Except for Column 79, the load applied to each column corresponded to its original sustained gravity load (1.0DL + 1.0SDL + 0.25LL) taken from the complete SAP2000 global model. The original load, prior to loss of floor slabs, was used due to the rapid nature of buckling and the fact that the lateral bracing provided by the collapsing floors would be lost prior to the load from that floor. For Column 79, however, the load was reduced based on the loss of the floor areas tributary to Girder 44–79 from the ground to Floor 13. Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report provide the detailed loads applied to each column. To reduce numerical instabilities, the loads in the second-order stability analyses were applied quasi-statically as time-histories with a linear ramp over one second and a plateau as well as 99% damping.

## C3.2.2 Section Properties

Each interior column consisted of a A572 Grade 50 W-shape which was in some cases, especially at lower floors, built up with side, web and/or flange plates of varying thicknesses. All reinforcing plates 2" thick or thinner were A36 steel; plates over 2" but less than 4" were A588 Grade 50 steel; and plates over 4" were A572 Grade 42 steel. Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report provide the frame sections along the height of each column.

# C3.2.3 Base Support Conditions

The bases of the Column 79, 80 and 81 models were assumed to be pinned at grade because the base details provided did not allow sufficient rotational fixity for it to be considered partially or fully fixed.

# C3.2.4 Bracing Conditions

The bracing condition used for each column stability analysis corresponded to the sequence of collapse established in Section C3.1 based on the configuration and characteristics of the bracing connections. Figure C3.1 provides the bracing condition of the columns in accordance with this sequence.

In these figures, the highlighted yellow connections represent either seated or header-type connections, which were conservatively assumed to have unlimited tension and compression capacity for the purposes of the stability analyses and were therefore modeled with linear springs with a high stiffness of 100,000 kips/in, which allowed a similar bracing condition to that provided by the intact floors in the global structural model (Earlier parametric studies which varied the spring stiffness showed that this value provided similar restraint to a pinned lateral support).

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In these figures, the red connections represent the axially-weak welded double-angle knife connections. These connections were for the most part modeled as nonlinear links with finite tension capacities in the stability analyses. The skew of the girders framing into Columns 79, 80 and 81 complicated the boundary conditions of the analyses because the two double-angle knife connections in each analysis were not orthogonal. As a simplification, nonlinear links were therefore assigned for the double-angle knife connections providing bracing to the column in its minor axis only because this is the direction in which the column was most likely to buckle. The double angle knife connections bracing the major axis of each column were assigned linear springs with no finite capacity.

The ends of the nonlinear links were restrained for displacements in the axial direction, and for all rotations. The links were defined so as to act in their axial direction only. The ultimate tension limit of these links was set to 0.85 kips/in of knife connection depth based on Dr Ingraffea's fracture analysis (see Appendix A of main summary report). This unit value of 0.85 kips/in was multiplied by the depth of each knife connection to determine the total tension capacity of each connection. The links were assumed to be infinitely strong in compression with a stiffness equal to the stiffness of the other springs in the model (k=100,000 kips/in).

Figure C3.2 shows the link force-displacement curve for a 14.5" knife connection, the tension side of which was generated from Dr Anthony Ingraffea's ANSYS 3D large displacement analysis which included vertical shear on the connection. To generate similar plots for different length knife connections, this plot was scaled according to the actual length of each connection divided by 14.5" on the basis that the stiffness and strength are proportional to the length of the connection. The scale factors and characteristics for each knife connection bracing an interior column are listed in the tables in Section C5.0.

Two analyses were run for Column 79 corresponding to Scenario A (floor failure initiating at Floor 13) and Scenario B (floor failure initiating at Floor 10) described in the main summary report.



Figure C3.1 Sketches of interior column bracing conditions as floor collapse progresses (collapsed slabs are indicated in orange and pink)

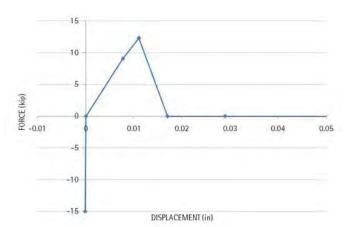


Figure C3.2 Axial force-displacement curve for a link corresponding to a 14.5"-long double-angle knife connection

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### C3.2.5 Initial Crookedness

Because the shape of the crookedness of each column immediately following adjacent floor collapse cannot be known, the stability analyses considered all possible crookedness configurations within the 1:500 slope limit described in Section C2.3.1. The demonstration that any one of these configurations caused instability of a column was sufficient to establish that the column would buckle. Therefore, to reduce the number of analysis iterations, the most critical out-of-straightness within the permitted erection tolerance limits was identified and only this case was run. This crookedness was imposed between column splice points. Otherwise the column elements were straight.

For each column, the selected crookedness corresponded in general form to the expected buckling shape of the column upon failure of links, which for each column was a half wave over the lower floors of the building. The specific location and height of the crookedness was determined using a series of linear buckling analyses in which lateral supports to the column were sequentially removed from the lowest vulnerable portion of each column until an eigenvalue corresponding to the first mode of buckling (ie "buckling factor") of less than 1.0 was produced, an indication that the column would not be able to support its sustained loads if it were to be unbraced over this height.

Once the height of the crookedness was established, the location and direction of the "kink" in the crookedness was selected to impose the most critical lateral force on the lateral bracing members in tension. This simulation provided a realistic representation of the lateral forces that may well have been exerted on the column's bracing elements as a result of its allowable out-of-straightness. Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report show the crookedness used in each column stability analysis.

# C3.3 Interior Column Stability Analysis Results and Interpretation

The results of the stability analyses demonstrate that for each interior column considered, the lateral bracing that the column was forced to rely upon following an adjacent floor failure was insufficient to brace it under sustained loads. In each case, the lateral forces imposed on the weak welded double-angle knife connections under the sustained loads were large enough to sequentially fail, or "unzip", the column over a sufficient height to cause it to buckle. The specific results for each column are presented in Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report. The following is a summary of the general results of these studies.

### C3.3.1 Link Failures

The nonlinear links in the models began to fail under application of sustained service loads. In each case, the first link failure was followed immediately by the failure of adjacent links due to load redistribution, effectively "unzipping" the column over a certain height.

### C3.3.2 Column Forces and Deformations

For each column, the link failures progressed until a sufficient number had failed to cause uncontrolled increases in lateral deformation and bending moments as a result of P-Delta effects.

Because the analysis was not set up to capture the inelastic material behavior of the column itself, the results of the analyses are valid only until the column cross-section reaches its yield stress at its extremities due to the combined effects of axial compression and bending moment. This point is taken as the buckling point and the end of the analysis because the areas where the column's cross-section has reached the yield plateau have zero stiffness (E=0) and are no longer able to provide resistance to the bending forces inherent to buckling. Therefore a smaller cross-sectional area is left to resist the same bending forces, causing a rapid deterioration of the stability of the column, or inelastic buckling (Ref 18).

To determine the yield (or buckling) point for each column, the maximum resultant stress (P/A + Mc/I) for each time step and for each cross-sectional type over the buckled height was calculated. The first point at which the maximum resultant stress at any location in the column exceeded the strength of the steel was taken as the buckling point. The steel strengths used for these calculations corresponded to the averaged values determined from available mill test reports and were therefore higher than the standard design strength values. Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report provide the stress calculations for each column.

# C3.3.3 Data Output

Result plots were generated to illustrate the onset of buckling of the columns under their sustained loading. A plot was generated for each column showing the maximum lateral displacement versus load step. Figures 5.17, 5.18, 5.20 and 5.38 of the main summary report provide the plots for each column analysis. The plots show that the lateral displacements increase exponentially as the analysis progresses. At the onset of yield in the column, the slope of the curve has significantly increased, indicating instability of the structure.

# C4.0 ADDITIONAL STABILITY STUDIES

Supplementary stability analyses were conducted on the eastern interior columns of the WTC7 structure in order to further substantiate that the non-code compliant lateral bracing of interior columns was a principal factor in the global collapse.

## C4.1 Evaluation of Column Stability with 2% Bracing Provided

Stability analyses using design loading (taken from the SAP2000 global model, including reduced live loading) with the same methodology as described in Section 3.2 were run for Columns 79, 80 and 81 using the same assumptions regarding adjacent floor failures. However, instead of using the actual capacities of the lateral bracing, the links were increased to provide either 1 or 2% of the design loads in each column at each level depending upon the number of sides on which the column was assumed to be braced. Figures 5.19, 5.21 and 5.39 in the main summary report present the primary input parameters used in the analyses as well as the results. Only Scenario A for Column 79 was considered because a demonstration of structural stability for Scenario A (ie floor failure initiation at Floor 13) implies structural stability of Scenario B (ie floor failure initiation at Floor 10).

The results presented in Figures 5.19, 5.21 and 5.39 of the main summary report show that after application of the full design loads on the columns, no links have failed and as a result, the bending moments in the columns are very low. The displacement plots which increase linearly as the load is applied and then stabilize illustrate that the columns are stable.

C5.0 INTERIOR COLUMN CONNECTION TYPE CATALOGUE

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117734 U	952246	State	Cechalli	311	0.79	W1601	foh	8000	8.5	950	W21s84	Header	Us/2/676 best-15/2/6716 best	415						٠
Approximate the same of the sa	W2 total	Cult	1600%	11.5	629	W16G1	-	Lechtiff	98	0.59	W24s76	Header	L4s2x5/16 best-15x3x5/16 best	115		,				,
MACS	W23x64	Colfe	L4x3x3/8	118	679	W1631	Code	Lechalle	50	0.58	W21s44	Header	L4x3x5/16 best-L5x3x5/16 best	115						
the see	9923944	Cole	LACHTE	118	626	W16/23	Code	160038	9.5	650	W24e76	Heater	(4005/16 best-15c2c5/16 best	115					1	5
WHITE STATE OF	W21s44	Knife	24x3x3/8	115	44.0	WEGST	Koalis.	\$64.94E	8.5	0.59	W21544	Header	L4x2x576 best-L5x3x576 best	1115				2.2		0
1000000	W2 1664	Kriste	L4x3x3/III	311	620	W10:01	Knife	\$4x2x310	38	0.59	W24x76	Header	LACASSTO bent-Liki July 16 bent	311	*	*	.2.			90
	362 tres	Sec.	LACZCITE	11.5	626	W10/31	4my	(4e/pc/3))	58	-0.58	9421944 4421944	Header	Uschights bent-Uschight bent	211	24					1
10000.000	W2160	Cush	LAczezy®	11.5	629	W1805	Costs	L4x2x39	8.5	659	W24x76	Header	LAX2nG16 best-LikzleG16 best	11.5	æ	æ	ं			
W14x211	W2160	Cute	(4c3c3/8)	11.5	0.73	Wrands		160.003	8.5	0.58	WZ1NSQ	Header	L4x2x4/16 best-L5x2x5/16 best	311	*	+2	÷	*		*
(Marketon)	W2560	Knite	Likehellik	11.5	620	Wthos	Kn5	1442438	9.5	0.69	W24474	Header	L4x2x5/16 best-L5x3x5/16 best	211						9
11 145,745	W2 5x64	Cs4e	L4x3x3/B	11.5	64.0	W1601	Costs	Lechning	8.5	620	W21544	Header	L4x3x5/16 best-L5x3x5/16 best	115		*				ä÷:
The same of the sa	W21e44	Cult	L4x3x3/B	11.5	60.0	W1631	Code	14-2-28	8.5	650	W24s26	Header	LASSESTIE best-Lindely16 best	11.5	**			ŧ		5
W34KJ70	W23444	Colfe	CACALIN	511	679	W1601	Code	1603.03	8.5	0.58	W23544	Heater	LAGSG16 best-LagsG16 best	311	17	17	17	61		H
101.797-000	W23544	Knft	(Achd)8	11.5	629	W1903	Keste	Lechtille	8.5	0.09	W24:75	Header	L4x246/16 best-L5x2x5/16 best	11.5		*	*			
0.0776.00	W2 test	Crite	LACROIDS	11.5	62.0	W1601	Coste	(NC) (TAC)	88	0.59	W21s44	Hesder	United 11s bent-United 116 bent	11.5			200			
NAME OF TAXABLE PARTY.	W25e44	Krife	L4c2c3/8	11.5	670	W16d1		(4c2c30)	8.8	0.58	W24s26	Header	[4x3x5/16 best-13x3x5/16 bett	11.5	22.	22.	725		_	
0.00000	W2 1464	Colfe	U60308	11.6	620	W1601	Code	16/3/38	88	650	W21966	Header	L4x345/16 best-L5x345/16 best	11.5		*	*		_	٠
MYANGOE	W21c44	Cuite	LACACAD	11.5	620	W16G1		(4000)	9.5	0.59	W24s76	Header	Ukide(716 best-Estal/216 best	11.5				2.5	_	
	W21x64	Code	14000	11.5	0.79	W1601		LeO-300	979	0.19	W21s64	Header	U4026216 best-U6026216 best	111.5	2				_	
W14e455	W21e44	Crafe	(4c)c)()	11.5	620	W16c31	-	(4c2c3))	9.0	0.09	W24x76	Header	[4434276 best-Un34276 best	11.5					$\rightarrow$	
200000	W21s44	Knife	(4c2c)/8	115	44.0	W1601		(4/9/3)	385	0.59	W21s44	Header	EAChST6 best-Unit-S16 best	311			2			2
Wiketoo	W2 1464	Krife	L4x2x3/B	11.5	0.79	W10-01		(4c/c)))	4.5	0.59	W24x76	Header	LACKSTO best-Uschilff Chest	11.5				20		2
	W21x66	Colfe	LACZCITE	11.6	62.0	W10-01	Code	(44243)	9.6	0.59	W21944	Header	United the best-United (Chief	11.5						
Sits on Transfer	30			0.	35	35	25	336	(it	35	35	y	25		35	25	35	3.5		35
15	+:	*		10	+:			+3	10			1		70	*	+1	*0			+3
3,			.*	. 9	0.	0.		.00.		4.		-								0
ı	*	æ		œ.	3	æ	es.		9	æ	e.	v	38		÷	÷		38		98
+3	43		t)	13	-	9	65	32				-		12			- 23			9
	3	1	(it	Œ	51	+	÷	120	(it	+	*	i t		÷		77	it.	51		SŤ
Scale factor	listed for t	each knife co	onnection correspon	ds to ratio	of depth	to the 14	.5" deep con.	Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	araffea for	axial capa	city and s	tiffness								

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																		Ments		
A. B	-	-		100000				2	1				1	10000			-			
The audit	NAME OF TAXABLE PARTY.	100	14-3-16	Total redicare	7 M. 100	30.00	-	10.000	Profession and Profes	18,00	The state of	100	140,414	Part and state	races	Mean	200	10-0-06	Pull suffices	1800
No.	100000	100	(45)-30)		400	the same	-	100,000		440	OF SAME	O. C. C.	The Participant of Street, and			*******	Company	2000000		
W14490	116631	Code	(443438)	98	659	W24e62	-	(64,91)	311	0.78	W22484	Header	(4x4x5/16 bent	273						1
-	WHEGH	Cole	14x3x3/8	88	650	W24e62	H	techilli	115	629	WOTEGO	Header	[443544/16 best-[5x3545/16 best	11.5		,	10			1
W14c99	W16.03	Colfe	L4x3x3/lb	8.5	0.59	W24H62	Cooks	Lech 310	11.5	67.0	W21550	Header	[4e35st/16 best-Us25s916 best	11.5	30				ж	2.5
and a second	10908	Code	Like Section	88	450	W24e52	-	(600)	11.5	62.0	W2155	Header	(Anthe)16 best-(Sche)16 best	11.5						*
W14x109	WHEGH	Co.Se	14x3x1/8	9.8	650	W24e62	Knite	\$6.00 M	1115	67.0	W2150	Header	LANDSTE best-Lisbest 6 best	311				300	æ	2
1004-1000	W16.31	Kride	LACKARIN	912	65:0	W24462	Coste	(44.34.30)	115.	0.78	W2150	Header	L4x3x5/16 best-L5x3x5/16 best	311	*	18	*	30	30	180
M14x132	W16/21	Code	1443433	8.5	0.58	W24e62	Cole	16/3/33	115	828	W2150	Header	L4sh4215 best-L5sh4316 best	115			,			1
100.00	10900	Kos%.	L4x3x3/B	88	650	W24H2	Kra3e	14c)x30	115	678	W21450	Heider	L4x2x5/15 best-L5x3x5/16 best	115	35	æ		3.6	28	8.
MINKE IN	W1601	Colfe	(44.34.3/8)	8.5	650	W24s42	Cish	(600)	311	0.78	W2160	Header	L4x2x4216 best-L5x2x5716 best	11.5			1	3		*
March and	W16d1	Code	UADAIR	98	650	W24x62	Cale	180701	115	609	W21450	Header	LAGSC/16 best-LSc2cs/16 best	115						*
W14K333	W18G1	Code	14c3c3/8	38	650	W24e62	Code	(40.00)	115	62.0	W2150	Header	[4034576 best-1543476 best	115				*	æ	.1.
the Cont	W16d1	Colt	L4x2x3/8	R.S.	650	W24e62	Cult	(60.00)	118	0.79	W21560	Header	LACHSTHI best-LEGISSTG best	11.5				5	9	*
W1462D	Wieds	Knife	1/DD41	918	650	W24x62	Code	(techile	115	0.79	W2150	Header	Uschig16 best-Uschig16 best	115		:			×	3
TOTAL STORY	W16/31	Chile	14/2/30	9.0	050	W24452	Crafe	00000	115	0.78	W2150	Heater	L40.045/16 best-Us-24/316 best	115	*			*	*	*
W166233	Wiedt	Colle	LACION	8.5	0.59	W24e62	Costs	Lectural	115	628	W21150	Header	Lincolatific bent-Uschight Grent	115			,			
	WISG1	Code	technile	50 80	0.58	Works	Colt	8(6/24)	115	679	W2160	Header	Linitial 16 best-US 205/16 best	115						1
n Head	10914	Suite	(4x3x3)ll	98	850	W24e52	foh.	(60.03)	118	820	W2160	Header	: UACHETTS best-LScheFTG best	415			,			
2000 00000	W16G1	Knik	DARKER	8.5	650	WSteeds	Kah	Lechtiffs	11.5	0.79	W2150	Header	LASASZ16 best-EstaSZ16 best	115		,			,	1
W14c283	WT6.31	Chile	L4x3x3/8	38	650	W24e62	Crite	Lechalls	115	67.0	W2150	Header	LANDISTE best-LSchistlife best	115	٠					*
1777	10900	Code	14-3-38	8.5	450	W24e62	Cole	(600)	115	67.0	W2150	Header	(Achights best-Usrais/16 best	11.5						*
WINESTE	W16G1	Kn.	24:3-1/B	9.8	450	W16-77	Colfe	\$64.93B	115	84.0	W2150	Header	14x2h5716 best-15x3h5/16 best	911			2	2.2	0	2
100	W16d1	Kride	L4x3x3/III	4.6	620	W24402	Krafe	16/2/30	311	0.79	W2160	Header	L4x245/16 best-15x245/16 best	311	8		,2,	283	20	.2.
10 145 A47	16914	Colte	Ukracilii	40	650	W24e62	to)	(technii)	11.5	92.0	W2150	Header	UACHG116 bent-Uschig116 bent	211						-
ORCHANIS.	W18G5	Coste	LikeSkSpB	8.5	62.0	Wzkeśż	Crafe	Like Strips	11.5	629	W2560	Header	L4x2x5/16 best-L5x2x5/16 best	11.5	es .	æ	i e			*
COPY MAN TO	WHILES	Colfe	L4x.3x378	8.5	650	W24e62	Kin fe	56200	311	0.28	W2159	Header	L4x2x5/16 best-L5x2x5/16 best	311	*	*	*	**	*	*2
00,746,000	W18k25	Kn3r	Like he tilling	8.6	650	W16/37	Sm. Sm. Sm.	160333	11.5	629	WOTHER	Header	L4x2x4/16 best-L5x3x5/16 best	11.5		+				9.
	W16.01	Code	L4x3x3/3	38	620	W24662	Crafe	Lec2v3/8	11.5	62.0	W21x50	Header	L4x3x5/16 best-L5x3x5/16 best	115		*			.+	1
(ACCALANCE	W16/21	Code	L4x3x3/8	3.0	650	W24e62	Kesh	1642438	118	0.78	W2150	Header	L4x3x5/16 bent-L5x3x5/16 best	11.5	- 45	- 40	*	**	5	5
Distance of	W16G1	Colfe	£4x2x1/8	8.5	950	W24462	Code	160533	211	629	W2150	Header	LACASTS best-Linds/10 best	311	i i	i t		57	i d	7
W16x455	W1601	Knite	[4c2c3/8	8.6	650	W24x62		14c3c38	115	0.78	W21x50	Header	Local/16 best-Lischs/16 best	115		*	8	*		÷
TOWNS OF THE PERSON OF THE PER	W16d1	Crite	DACKUM	8.5	0.59	W24=62		Lechellis	11.5	0.29	W21450	Hesder	L4x2x5/16 best-L5x2x5/16 best	11.5					c	*
0034900	W16/21	Coste	(4x3x3/0)	912	650	W24e62		(4c3x)))	11.5	0.79	W2160	Heade	LAGSENTE best-UnDel/10 bett	311	2	æ	12.	7	2	2
	761601	Colfe	00000	8.6	0.59	W24e(2	Ente	DECEMBE .	116.	0.79	W21450	Header	CACONSTITUTE DATA LEGISLATION DELL	11.5		*				*
WYANGO	W1601	Colle	L4x2x3/ll	9.0	650	W24e62	Cale	(600)	115	0.79	Withfo	Header	Undergrif best-Uschelzfild best	11.5				2.5		*
	W16.01	Code	140000	9.0	650	W24x62	Civile	(feO+3))	115	67.0	W21450	Header	140205716 best-15c2n5716 best	11.5	æ	1.0		2.	×	2
96144/80	10010	Code	L4c2c3/8	3.6	650	W24m62	Code	(4c3c3))	118	0.78	WZDSO	Header	1403/U16 best-13c3/U16 best	11.5				6		0
	W16G1	Knife	U4241/8	8.5	620	W24e62	Knite	(60.00)	11.5	67.0	W21550	Header	LAXASTIS best-Uschistlichet	511			12	500	2	
Witness	10011	Krife	L4x2x3/III	4.6	650	W24+62		16/2/10	415	0.79	W21450	Header	L4x2x5/16 best-L5x2x5/16 best	11.5	(3)	(4)	8		20	*
	10000	Code	14x2x3/8	40	650	W16477	tes	(44243)	11.5	0.79	W21450	Header	LANDSTR bent-Underfold bent	11.5						
Opportug.	WT8k35	Co.Se	L4x2x3/B	38	650	W24=94	Colt	(4c/2)/0	2.02	123	W24x68	Heider	14r35r5/16 best-(5r35r5/16 best	115	25				30	3
A00000	Withest	Crite	C4r2r3/III	11.5	120	Withes	Confe	(600)	311	62.0	William	Header	LANKS SINCE	4.5	*	W14x30	Healer	55/29/276	185	1.0
Witness	W21s84	Kn3s	L4r.3r33	14.6	100	W30x113	the Contract	(649.33)	22.5	140	W30x173	Header	(Andritz)	38.0						
	W21e44	Code	[4x3x1/8	11.5	44.0	W20:69		14x2x30	34.5	1.00	W12x19	Heider	No information Available	*	4	÷	3.8	:X:	96	÷
Wheeler	W24el2	Code	(4x3x3/III)	r.	1.45	W36-34	Cash	16/2/20	n	1.06	W32k118	Header	9.25c339.bmt	343		*	- 63	9	9	*
	WORLD	Kris.	5403010	**	0.61	MATERIA AND	,	14.5.00	-	0.000		100000	10.00.000	1000						

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity

Where the beam size is indicated no member is farming into the count of an area of a manual steel shop drawing for that connection

The information specified for each connection was taken from the datest corresponding Frankel Steel Limited steel shop drawing for that connection

The information specified for each connection was taken from the dates from the consequence of the connection of the con

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Column	Beam	Type	204	Length Bell	Factor <sup>†</sup>	Beam	1 ppe	Spe	Length Sell	factor	Beam	Type	支	Injultini	factor?	Beant	lype	504	Length brid	Factor?
- Anna market	,			P	à	WF6/26		Lechtin	8.5	150	WTG-26	Header	UNNSTR	11.5	-	WESSTR	Heater	Changite	85	÷
OCCUPATION OF THE PERSON	W24x68	Corte	1442438	11.5	0.79	W1840		1443438	11.5	629	W3069	Header	No information Available		94	W30x108	Header	13434516	34.5	G.
80-913	1812/16	Yesh	(4cm38	9.6	550	WISHO	Krafe	24c2c3t38	115	200	W27584	Header	1442438	31	+	W25c161	Header	14:01575	19	
	WHENED	fate	(4c2c3)	27.5	171	M1940	Code	1447436	311.5	679	W33x130	Header	Dept5	34		WOOM 108	Header	(46263)3	18	•
W146132	W24x55	Seated		+		WISHO	Krife	(4c)(dg	11.5	0.73	W21550	Header	100676	11.5	٠	W27x161	Header	L3v3v38	17.5	្
Carried a	W24455	States		ř	7.7	W12e40	Knife	144,3438	115	679	W21540	Header	13/09/216	11.5	r.	WDARS	Header	13/0/63/6	11.5	ê
10146/109	W24x55	Seated	(X	111		W12640	-	14636303	111.5	679	W21x50	Header	DOMESTIC	11.5	٠	W2hctst	Header	130038	17.5	٠
	W24655	Seated			4	W18p40	-	(40)(3)	311.6	0.79	W21400	Header	0.00000	511		762465	Header	0.00476	11.6	
WALTER	W24x55	Seabed	(14	4	- 17	WEST	Krite	140039	115	62.0	W2356	Header	03/04/216	115	, e	W23c161	Header	13/2/01/8	17.5	70
	W24455	Stated	*	i i	Ŧ	W15-40	Keife	\$6(3F)B	11.5	62.0	W21550	Header	0.006716	115	2	W24455	Heate	0.00576	31.5	+
Wide/201	W24655	Seated	20	-	÷	W15e40	Keilt	LACHTR	115	67.0	W21550	Header	Udegie	11.5	÷	W23c161	Healer	130008	17.5	£
	W24655	Seated	12			W13=40	Knike	(4chill)	115	62.0	W2160	Header	0.006/16	115	**	W24655	Header	13/04/16	11.5	+
965-100	W2455	Stated		4	- 43	WTSHO	Krite	TAKINDS	115	67.0	W21950	Header	13/0/6/16	115	F	WZhcf61	Header	DOME	17.5	
117700141	W24455	Seated		ï	¥	Without	Khite	140,438	11.5	628	W2158	Header	31,54013	1115	8	WHEE	Header	31,510.03	11.5	ř
2007-9500	W24655	Seated				WISHG	Keife	Depths	3.11.5	6.73	W21450	Header	01/24/210	211	96	WZPc161	Header	Unchalls	541	*
797 W M	W2465	Search	9			WISHO	Knite	(Achill)	11.5	200	W2150	Header	Changie	115		35456	Header	13434676	11.5	*
and and	W24x55	Seated		4	,	W1386	Kryb	\$6.0c.35	11.5	670	W2150	Header	372053	511		W2hr161	Header	Chadit	17.5	
1136347	W24x55	Septe	*	ī	Ŧ	Willes	Knife	(4e.3e.3)))	11.5	679	WZ1560	Header	01/9/010	11.5	÷	30,4455	Header	DOMEN	211.5	+
000000000	W24x55	States			1	W15e40	Knife	\$44.74.38	113	620	W2156	Header	13×3×216	11.5		W22c161	Header	Ch-2x38	12.5	
O Company	924655	Seated		-		W12540	Krife	(4c3c3)	211.5	62.0	W21x50	Header	0.00676	11.5	*	303465	Header	13434516	11.5	
1014-310	WZ4655	Seated		4		WEST	Cocke	(600)	115	62.0	W2350	Header	Disperte	11.5		W23k161	Header	Chacin	17.5	+
100000	W2465	Seated		4	- 40	WISHO	Kuite	(4e.b.)(b)	115	67.0	W2156	Header	Dx26216	115		W2465	Header	13/04/16	115	
3039793	W24655	Seared		7	7	Willead		14c3x38	11.5	62.0	W2350	Header	01/9/07/0	115	4	W23x161	Header	Lbcbclik	37.5	+
	W24455	Seated		-	12.	WIEseo	H	SECOND.	115	62.0	W2150	Header	DOME	11.5	6	W/465	Header	1304%	11.5	
330-9530	W24655	Seattle		4	-	WISHO	Kulle	DACADIS	11.5	62.0	W21650	Header	13/06/16	11.5		W23x161	Header	DOMB	17.5	+
	WZ4655	Seated		ų.		WISHO	Knife	1407038	115	673	W21550	Header	0.006716	115	Þ	W2465	Header	Drawfile	11.5	
WINNERDO	W24nSS	Seated				W18e40	Krafe	(44.343)	11.5	0.78	W24955	Header	13/04/16	11.6	•	W27s161	No actor	(303038	17.6	÷
	W24d5	Seated		i	٠	WTSHEO	Keife	1400038	311.5	0.79	W24655.	Header	13/04/216	11.6	4	W24658	Header	13/04/16	11.5	
W34e550	W24x55	Some	*	-	÷	WTS-40		(4chr)))	115	679	W74655	Header	DOMETE	116	+	W22ktft	Header	University	17.5	+
Carrier C	W24455	Zeneri Teneri	12	ï		W15=40	-	tech 10	115	200	W2150	Header	DAMATE	411	,	WiteSS	Header	U3-0-4216	343	+
2014x625	W24065	Same	7	ī	Ŧ	WIBSO	KNF	(4e3x3)))	11.5	67.9	W21560	Header	13-09-67-0	11.5	÷	W27x161	Header	Christill	27.2	+
	W24m55	States.				W12840	Name of	144713	113	0.3	Watskie	Header	DOM216	112		MONESS	Neader	0.00476	11.6	1
W14e605	87,6053	Same S		•		WIESEO.	+	(4e.pr.))s	9 1	500	05417M	Header	District I	911	,	W2/3/83	The age.	Chestallia 19-0-004	1/2	
	Widness	- Control				William	Total Control	14/4/18	446	0.74	WORKS	Header	1309636	all a		WORKER	Bade	Debell	418	1
WHeeles	W24x55	Same	×			WISHO	Kinfe	140)(1)3	11.5	67.0	W2150	Header	LDOGGE	11.6		Wzaes	Header	130638	11.5	•
and the	W2465	Seated	-			WENG	Culte	tenis	115	67.0	954.DM	Header	30696	11.5		WZ7k163	Header	(DOOR)	17.5	
11 41000	W2665	Seated		4	i i	WISHO	Knife	LACALIE	11.5	62.0	W21550	Header	Lha6/16	1115	14	W2465	Header	District	11.5	4
00146730	W24x55	Seated		4	40	W18e40	Knife	(40)(3))	311.6	0.79	W21x60	Header	DOM/016	11.5	93	W22x161	Header	United98	3.0	*
100000	WZ4nSS	Seated	4	4		WISHO	Kulk	(4e.br3))	11.5	82.0	W21x50	Header	13/04/16	311		1024455	Header	13/04/16	11.6	è
W14c730	W2465	Seated				W15e40	Keife	DADOR	11.5	620	W21450	Header	0.00676	577		W2hci61	Header	130303	17.5	
	W24e55	Seated		T.		W1540	Knite	LACK IN	115	679	W21560	Header	Undelite	118	÷	W2465	Header	130616	115	+
	M5-27	Enth	(2)44533	29.5	272	W146342	Trans One			92	Walktoli	Header	0.000.00	22.5		W27x16T	Header	13/04/216	18.5	
W14e500	W21s44	Corte	(44343)))	11.5	0.79	W14s342	Trass One	3.0		ž.	W21560	Header	04/346/10	#	٠	7674476	Header	[4x4x]]]	34.5	٠
	W24e68	Krafe	14(303))	115	0.79	W14x342	Teats One	ř.	,		WZ4kB4	Header	14094316	18.5	£.	Wilkitla	Heater	(4s-4s-318	23.5	j.
W14x93	W27584	Corke	1463638	34.5	100	Wthits	KNY	(4c3c)(8	85	850	W2254	Header	13×3×5/16	30.5	*	W33k118	Header	13404516	23.5	
Sits on enstring Col	,	à.º	- 0			-		4		:					,					*

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity
2. Where no beam size is indicated no member is farming into the count of the connection and the formation specified for each connection was taken from the latest corresponding Fankel Steel Limited steel shop drawing for that connection
3. The information specified for each connection was taken from the latest corresponding Fankel Steel Limited steel shop drawing for that connection

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			North					South					East					West		
Column	Stein	- Libe	No.	length (in)	Factor?	Beam	Type	5.	Limphibing	Factor?	Sean	Type	5.	Length [m]	Factor?	Eram	fype	155	[withful	Factor
W14c50	WTBe60	Code	LACASIN	115	62.0	W24m62	3 Header	1543 5438	405		W33x130	Header	54×3×3/9	36.5	1.60	WOOSS	Healer	L4chs216	242	5
100.000	WYBest	Code	(4x3x3/8)	14.5	100	WINDS	D Krish	1642430	3.5	0.53	W24s76	Colfe	54c2c39	14.5	1.00	W24s76	Healtr	(4x3xf/16 best-Us/3xf/10 best	11.5	::
03400	WYBeso	Chile	£4x3x3/8	311	620	W12x19	total Code	(64.34.3))	85	850	W18640	Ent	5443439	115	620	W21544	Header	DAD4916	8.5	*
101.001	W18640	555	L4x3x3/III	111.5	620	Wthth	Solt Cole	Lechilli	9.6	0.59	W1840	Snite	14<5<30	115	62.0	W24-76	Header	(4x3x5/16 bent-(5x3x5/16 bent	11.5	.1
	W18e40	Code	14x3x3/8	311	0.79	W12c19		16/25/30	8.5	6.63	W18e40	Coff	64c3c30	11.5	620	W21s44	Heater	(4x3x5/16 Sent-Lin3x5/16 Sent	115	*
00.140.100	W18e60	Code	14434378	118	620	Wthtts	Code	16/3/38	9.5	950	Willes	Coult	1443438	115	67.0	W24s76	Header	Me behavior to Available	* 3	*
	WIREGO	K845	24x3x3/8	111.5	620	Wthts	See Keek	14/3/38	8.5	6.68	WHENG	Colle	24-D-39	115	0.79	W21544	Header	(4x2x5/16 bese-15x2x5/16 best	1115	2
1000-000	WIBNEO	Krish	L4x3x3/III	315	620	W12x19		L4sr2s300	385	0.58	:W18460	Knth.	54/3/30	11.5	0.79	W24s76	Header	Na Information Available	*	*
771 201 14	WIBNED	Crite	144,34338	311	82.0	W12x19	e Code	14/3/33	8.5	0.58	WIBMO	Colb	147378	115	0.79	W21s44	Healer	Likeling/16 bene (Selhighlis bent	115	
10000-000	W18e40	Ku5k	14×3×3/0	11.5	679	Wthth	450 E	14c3/30	8.5	0.58	W18e40	Cook	54cb39	11.5	0.79	W24s76	Heater	Na Information Available	34.	3.
10/46/45	W18e40	Colfe	(44.34.37)	311	0.78	W12x19	910 6	teach	9.6	950	Willesto	Colt	14000	11.5	0.79	W21s44	Header	LAGREFIS Beet LEADS/10 best	11.5	
	WTBe60	Code	140,3438	11.5	620	Wthers	6 Cale	160.038	85	0.59	WYBARD	Cale	060000	115	629	W2406	Heador	LAUNG16 bere LSch 916 best	115	
W14c155	W18e40	Code	(4c)c)lill	115	620	Wt2ct9	9 Cult	(40.31)8	8.5	650	W1840	Colle	(44343)	115	620	WINE	Header	LAUNE/16 bert-Lichtly16 best	11.5	.1.
200000000000000000000000000000000000000	W18e40	Cole	14x3x3/8	115	0.79	Wthitle	-	(40.303)	115	0.59	Wtheo	Code	(4chill)	11.5	0.79	W24s76	Header	LAXDS718 bent-Us/3/55/16 bent	115	
W14x136	WIBNEO	Knik	1402df8	11.5	0.79	W12x19		9(0)(0)(1)	8.5	0.59	WIBNO	Colle	Uch39	11.5	620	W2 ts44	Heafer	L6x2x5/16 bert-15x2x5/16 best	115	3
Management	WTBMG	Cosh	Lechtlife	115	620	W12x19	D Kriste	0(429)	8.5	050	W1840	Colle	24(3)(9)	115.	620	W24s76	Header	L4x355/18 8445-Us-205/16 bent	115	*
W16x383	WYBe60	619	LACION	115	67.0	Wt2n19	- Code	(80.00)	8.5	650	WIBMO	Colt	(4c)c)()	115	0.79	W21s44	Healer	LACING 15 best-LSc2xS16 best	115	
	W18e40	Suite.	Lite/2x3/6	115	628	W12x19	H	8(5)(2)(3)	58	0.59	WTBMO	Colle	(44263)	115	0.79	W24s76	Header	Lechel/16 benk-Endel/16 bent	11.5	3
M14x211	WYBed	508	Sec. 21.0	111.5	620	Wt2st9	493 E	16-3-39	8.5	950	Wffee	Code	(4424)9	415	620	W21s44	Header	LAGNS/16 been Unans/16 bent	115	
sales and	WTSe40	Knik	140.00%	115	620	Wt2s19	fah.	Lechalli	8.5	0.59	Withset	Snife	(4cb3)8	115	620	WOACIE	Header	14/3/5/10 best-15/3/5/16 best	11.5	
V 146,233	WYSMED	Cnite	L4x3x3/B	111.5	628	W12x19	9 Cork	Lechalls	85	0.58	WTBHO	Corte	Lechals	115	0.73	W21s44	Header	L843h5/16 bests (5x3h5/16 best	111.5	٠
the same	WTSec	Code	144.34378	115	620	Withh	4 Code	(463038	8.5	950	W1840	Crete	\$4x3x3/8	115	67.9	W24s76	Healer	Lkxbs/35 best-L5r3x5/16 best	115	*:
MINKS.	Withsto	Knis	8/14/49	11.5	420	W12:19	WANT O	14-7-38	85	0.59	WHENG	Colle	24ch399	115	6.00	W21s44	Heaftr	(44245/16 bene (54245/16 bent	511	
100-700	W18e60	Krish.	L4x3x3/III	315	620	W12x19	South Confe	16:2:30	3.5	62.0	W1840	Knite	54x3x30	211	62.0	W24x76	Header	L4x3x5/36 benz-15x3x5/16 bent	11.5	(2)
	3978e60	Colt	L4x2x378	11.5	626	Wibrits		(4c.2c.33)	9.5	-0.58	Williamo	Cotte	L4c2c3/9	11.5	0.75	W21544	Heater	C4x2x5/15 best-Us/2x5/16 best	115	
110,000	9813640	Park.	Like 24-378	11.5	0.29	Wt2x19		Lechale	8.5	650	W2 tek4	Knite	1443439	11.5	620	W24x76	Header	Ukr2h5/16 best-Ukr2h5/16 best	11.5	*
	W18640	516	(4x.3x.378)	11.5	0.73	Wi2xi9	-	56208	8.5	0.53	W21s44	Engle	(44343)	11.5	0.75	W21650	Healer	(4x3±5/16 Seet-15x3x5/10 Sent	11.5	*
W14x342	WTBMCO	Cn*	Like he hills	115	640	WESTS	-	techalls	9.5	0.69	W21584	Ent.	(4cbc)B	11.5	623	W24s76	Header	(4x2x6/15 bend-Us/2x6/16 bent	11.5	
	W18e60	\$100 €	(4x3x3/8)	11.5	67.0	W12x19	4	865/297	8.5	62.0	W18s40	Coste	(4c3c3)8	11.5	0.79	W25s44	Heater	[4x3x5/16 best-Lis2x5/16 best	11.5	*
W54x330	W18e40	Cult	Like Str. St. St.	11.5	67.0	Witerla		(4.5.3))	918	0.50	Willedo	Coult	[4c2:3]B	11.5	0.79	W24s76	Header	LAXDS/16 best-Linds/16 best	113	**
	W19e40	Knife	£4x3x3/8	511	679	Wibits		(60.00)	9.8	0.59	W18e40	Coole	54-3-29	511	0.79	W2 ts44	Header	LACINETES Seen LECTRES Seen	511	14
W14x288	W18e40	Knft	(4c)c)t)	11.5	0.79	Wt2s19		1403/38	8.5	650	W1840	Colt	(4c)x38	115	0.79	W24s76	Header	[4126]16 betc.[5225]16 bert	11.5	×
100000000000000000000000000000000000000	WTBe40	Cuite	DAZADI	118	67.0	WESTS		000000	8.5	0.59	W1840	forte	14c3c30	11.5	0.79	W21544	Header	LACINETTS Bent-LEGINSTIG bent	11.5	
MARKET	WIBMO	Krite	(4c3c3/8)	11.5	0.79	W12x19		(46.26.30)	9.8	0.58	W15e40	Colb	(4c)c)()	11.5	0.79	9/24/26	Healty	LAGDS/16 best-LSchS/16 best	11.5	2
	WISHED	Colfe	CACACIB	11.6	0.79	WISH	-	Lectors	8.5	0.69	WTBMG	Corfe	00000	11.5	0.79	W21s44	Header	LASTATTA Seed Like 2017 to Sent	11.5	*
WTANGO	W18e40	Knik	14×2×3/8	11.5	670	Wt2s19	fash (	(6003))	9.6	0.59	Withdo	South	1442030	11.5	62.0	W24676.	Heador	CAXDA[16 bent-L5xDa[16 bent	11.5	
	WTBe60	Cosh	(4c)c)t)	11.5	0.79	W12s19	6 Cuife	16(3/3)	9.0	0.59	W18e40	Code	(4c)(3)	11.5	660	W21984	Heater	(Ax2nf/16 bere-Un2nf/16 bert	11.5	*
Without Str.	WYSHGO	Code	14×2×378	11.5	0.79	WESTE	440 G	(40.04)(6	9.0	0.08	Withsto	Code	(4c2c3)9	11.5	0.79	W24s76	Healer	LAX26216 benz-Un2nS716 benz	11.5	()
	Withse	Knife	U4241/8	115	620	Wt2st9	Rosh (	(60.03))	88	950	WHB40	Colle	(4cb3))	115	0.79	W21s44	Healty	CACING/18 best-United/16 best	115	
W14m405	Without	Krish	L4x2x3/III	11.5	0.73	W12x19	Forth.	16000	4.5	0.00	W18460	fah	56.00.00	11.5	0.79	W24x76	Header	L4x35f716 best-L5x3x6/10 best	11.5	8
	W18e60	Knih.	Ukracitii	118	0.79	W12x19		(NOOM)	9.5	0.59	W1840	fuh	LAchchill	11.5	0.79	W21544	Heater	(Ashiffs best-Ushiffs best	115	
- TRUCK AN	W14x342	Co.Sr.	14x3x3/B	8.5	0.59	W10626		56/2/3/0	9.5	0.59	W2150	Knite (offset)	14c3c30	16.5	111	W24x68	Healtr	Lbds476	11.5	35
	W14x342	Trans-One	*0	70	+3	W2465	S Code	(600)	345	+	W21450	Code (offset)	1443439	11.5	0.79	W24x76	Healer	CDOMETE	14.5	*
01144717	W14x3x2	Truss Ces		3	٠	WHADS	S Code	14-3-28	9.6	0.58	WOTHER	Knille (pilline)	1443439	11.5	0.78	W30x173	Header	\$4444Z	3.5	.0
	W14422	Chile	[4x3x3/8]	6.5	659	W16-26	6 Gate	54x3x3/8	385	0.59	W24e62	Cook (offset)	C4<2×3/8	11.5	0.75	WS2st8	Header	13/3/6/16	55	i.t.
W34c311	WIGHTS	Costs	18×3×1/2-1/16 weld	×	1.4%	W14s22	Costs	(625)	8.5	0.50	W24455	Faitaffell	Pt 243/4	22		W12514	Header	13/24/216	6.5	*
	W10,3E	Kale	54×3×100	\$	0.63	10110031	Co.fr.	160 30 305	**	0.00	Marin and a	Total Adding	21.66410	94		Mrs G. Y.G.	200			

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity

Where no beans size is indicated non ember is farming into the colour a. The Information specified for each connection is framing into the colour properties and the steel shop drawing for that connection

The Information specified for each connection is framing into the latest corresponding Frankel Steel Limited steel shop drawing for that connection

The Information specified for each connection is framing into the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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			North					South					East					West		
Column	Beam	Type	焚	Length Ent	Factor	Beam	Type	克	Limpth Snd	Factor?	Bean	Type	5.	[within]	Factor?	Beam	fype	-55	[w] theat	Factor
Wredo	W24e62	Code	Lechtlin	11.5	62.0	W14x22	Header	LAnadata best-Lisastatabest	88		W21564	Header	(4524576	118		W24655	Header	Lbo4216	115	
Media	WTSee0	M-Conn		1		W14622	Header	14x2x5/16:bent	945		W24s76	Header	144346710	2.0	128	W2150	Header	L4x3x6/16 bent-L5x3x6/16 bent	11.5	38
orana a	W12x19	Shift.	SEASON	9.5	659	W36x150	Header	(4x2x5716	17.5		W24x62	Header	L4x245/16 bmrt	12	0	W2254	Header	Lier Skit/27 Sense - USA Skit/27 Sensit	215	*
Water	W12x19	Cole	techtilli	919	650	W16:31	Seated		4		WIBAGO	Header	LANASCI16 bent	9.5		W2150	Header	(4x3x5/16 bent-15x3x5/16 bent	415	.1
	W12x19	Knife	[4x3x3/]	8.5	0.59	W16-31	Seated	26	æ	28	W18e40	Header	L4x6x6/16 bent	8.5	35	W21450	Header	(4x2x5/16 Sent-Lin3x5/16 Sent	115	*
100.000	812518	Code	LACHOUSE	8.5	650	W16.33	Seated	43	+		W1840	Header	14+4+5/16	8.5	5	W21655	Healer	LAxibs/35 best-Usrans/16 best	115	*
10 140 100	WENTS	Knie	E4r3r3/8	9.8	650	WEGST	Seated		ž		WHENG	Header	1444-5/16	3.5		W21550	Header	L4x3x5/16 benc-L5x3x5/16 bent	11.5	2
10014-000	WIZES	Kride	L4x3x3/III	8.5	0.59	W1603	Stated	30	Ŧ	80	W18460	Header	[45456]10	45.	*	W21450	Header	14x3x5/16 bent-Un2x5/16 bent	11.5	150
777.201.00	WESTE	Crite	Las 3x 1/18	8.5	0.58	W16-01	Seared	(K)	E		WIBMO	Header	34,599991	98		W2160	Healor	Last NEW NEW NEW SANGES SENT	11.5	2
10.24-10.0	W12k19	KssNe	L4x3x3/B	38	650	W16:31	Stated		e e	38	W18e40	Header	14445/16	3.5	33.	W21450	Heatir	[4x3x5/16 bent-[5x3x5/16 bent	115	8.
in Hertid	W12c19	Colfe	(fe/hc)/h	8.5	650	W1601	Seated			v	Willesto	Header	14445916	570	35	W2160	Header	Lax2x5/76 bere L5x2x5/10 bent	11.5	*
1000000000	W12x19	Code	DADOR	98	650	W16031	Seated		-		WIBMO	Header	14040S16	8.5		W2150	Header	LAX35575 bene-15434576 bent	511	*
W196326	W12c19	Code	LACKER	38	650	W16.01	System	(3)	31	35	W1840	Header	14,46,5716	985	*	W2150	Header	[44315/16 bere-[5x315/16 best	115	.1.
Take Links	W12c19	Colt	L4x2x3/B	8.5	650	W1601	Seated				Willedo	Header	Lance STG	8.5		W21550	Header	LAXDSZ16 bent-Us/245/16 bent	11.5	5
40.146.133	9012019	Knife	MDCM	918	650	W1601	Seated				WIBNO	Header	LANAS/16	878	i s	W21450	Header	LAXING/16 bene-Linite(16 bent	115	3
10034-5111	WINE	Crafe	Ukracijili	9.0	650	W16/31	Stated	30	÷	*	W18460	Header	14×4×5/16	9.5	30	W21560	Header	LAXDS/16 bert-Units/16 bert	115	8
11779110	Wi2ct9	Code	LACTOR	8.5	650	W16.01	Seated		1		WIBMO	Header	145455716	8.5		W21560	Healer	(Astrof/16 best-Uschriff best	115	*
***************************************	W12k19	Code	DACASE	10	650	W16/31	Seated				WTBr40	Header	14565276	85		W21450	Heador	L443x5/16 bent-(St3x5/16 bent	511	3
0.148,733	WI2NS	Knik	Section.	9.8	650	W16/31	Seated				Writed	Header	34,54948)	8.5		W2150	Header	LAXDS/16 been Links/16 bent	511	*
1917 1011	WESSTS	Colt	1463630	919	650	W16:31	Seated				Williams	Header	04% Sept. 10	98		W21450	Header	(4x3xf/16 best-likith/216 best	115	
	W12e19	Knite	L4x3x3/8	8.5	0.58	W1631	Stated	(8)	96		WTSHO	Header	L4x4x5/16	8.5	*	W2150	Header	LAKINS/16 best-USKRISY16 best	11.5	*
the same of	812518	Code	Lécholys	85	450	W16/33	Seated	-	+		W1840	Header	14+4+5/16	8.5		W21655	Healer	LANDS/36 best-Usr3x5/16 best	115	*:
	WINES	Knife	54-3-1/6	38	450	WEGST	Seated			92	WHENG	Header	144645/16	3.5	20	W21550	Header	Carbights benealist Intil bent	511	10
161500333	W12x19	Kride	L4x3x3/III	4.6	650	W10:01	Seated	98	(4)	8	W18460	Header	14×4×6/10	45	3	W2160	Header	L4x3x5/36 bens-15x3x5/10 bent	11.5	(2)
	Willetty	Colte	Lechtliff	58	650	1001W	Seated	1	141	,	WIRKO	Header	14+4+5/16	88	5	W21450	Healer	CASTAS/16 bent-Undel/16 bent	511	
(0.14×342)	W12x19	Code	L4x3x3/B	8.8	62.0	W1631	Seated	ÇE.	×		W25s84	Header	14/26/216	11.5	æ	W2550	Header	LAXDS/16 best-LichtS/16 best	11.5	÷
	W12k19	Colfe	(4x3x3/l)	8.5	650	W10.01	Stated	*5	ri.	*	W21s64	Header	(4/3/6/16)	11.5	*	W2150	Header	(4x3±5/16 best-(5x3±5/16 best	11.5	<u>+0</u>
W34x320	W12×19	Kn3r	LACECTER	8.6	650	W16/31	Seated				W21584	Header	Lachelysic	11.5		W21450	Header	(442h6/16 bent-Us/2h6/16 bent	11.5	
	WENE	Code:	(4x3x3/3)	8.5	620	W16/31	Seated	. *	÷	٠	W18s40	Header	[4×6×6236	3.5	٠	W21950	Heater	Lex2x5/16 best-Lin2x5/16 best	11.5	*
WHOM	W12c19	Cult	L4x3x3/lb	9.5	650	W1631	Seated	*	ŧ.		Willego	Header	Lase-5710	8.5	**	W21550	Header	LAXDASTIS bent-Undustriff to bent	113	5
200000	Wibers	Knife	CACACITI	9.0	650	W1601	Seated		9	12	W1840	Header	14445/16	98	7	W21050	Healer	L4chg/16bes-L5chg/16bert	511	17
W16x426	Wt2st9	Knife	Lacheljill	8.5	650	W1901	Seated		2		W1840	Header	L4x4x5216	8.5	*	W21950	Header	Lexand/16 benz-Linans/16 bent	11.5	*
(6)	W12x19	Crite	LACACITIS	8.5	650	W1601	Seated	0	ě		Willes	Header	L4>4>5/16	8.5		W21950	Header	LAKING 16 best-Likithoff to best	115	
W160426	WI2st9	Crafe	(4626)/0	912	650	W16d1	Seated				W15e40	Header	1446-6/10	415	2.	W21H50	Header	LACKST 16 bene-LSc2nS/16 bent	11.5	2
	WINIS	Cult	06000	86	0.59	W1601	Seated	*	+		WISHG	Header	[4464976	8.5		W21450	Header	LANDSTREENSCREENSTREEN	115	*
WYANGE	W12x19	Knik	LACACIA	8.5	650	W1631	Seated		g		Williado	Header	L4x4x5/10	9.2		W21450	Header	(4x3x5/16 bent-Uschs/16 bent	11.5	
	W12c19	Code	(4c)c)(I)	8.5	0.09	W1601	Systed		×	æ	W18e40	Header	14×4×6/10	912	æ	W0160	Header	(Axinf/16 bere-Unit/16 bert	115	2
W14n4EE	Witerli	Code	(Achd)))	8.6	650	W16c31	Seated	*)	*		Willeso	Header	Laves6716	45		W21450	Header	(Account the bench Literal ATT 6 bent	11.5	
	Wtheth	Knife	CACACIAII	2.0	659	W1631	Seated		Œ	28	WHB40	Header	144646/16	500	2	W21450	Header	Cachiff Shee-Unit(76 Sect	511	
W14e500	W12k19	Krift	U4:3x3/III	4.6	0.59	W10.01	Stated		÷	8	W18460	Header	[45456]10	4.5	*	W21450	Header	LAXDEFFE best-Lite 245/16 best	11.5	ž.
	WENT	Knin K	Deadli	58	650	W10-01	Sympol			,	WTB40	Header	LANNAGTE	99		WZ1MG0	Header	(Ashif/16 best-Us/201/16 best	115	*
00349400	W1626	Cook	(4c)c)(8	8.5	0.69	W24655	Colt	5462030	115	0.78	W18-65	Header	[4x3x5/16	115	es.	W24x68	Healtr	144354216154354516	11.5	ð.
	Wzess	£	18 Set2	185	+:	W24455	£	98.642	501	٠	Wrands	Header	D04516	4.5	*10	W1845	Header	144430	4.5	*
W14×550	W18k25	Kn%	(4e/e/l)	8.5	650	W27484	Enth.	(600)	14.5	1.00	WD4x131	Header	(40403)	2		W30x173	Header	Sentrit2	24.5	*
	W16-01	Chife	L4c3c3fB	8.5	650	×	×		7.5	96	W21450	Header	UN265/16	115	*	WS2s18	Header	0.00676	99	it.
Wishous	W14c22	Cult	L4x2x3/B		080	W24655	Code	(600)	22	0.83	W24x76	Header	14-0-5710	**		W33x118	Header	9.25c335 bmt	X	5
	tales of the	10.10				the same of the same of	,							-						

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Wheels	0	luse. The																West		
				tel Miles	Pactur.	Beam	1, pre	See	Length Soll	factor	Beam	Type	5	Injulation	factor?	Beans	lype	Sar	Length Brid	Factor
		***		6	1	W16/26	Khite	1403438	85	Di co	WTSe26	Header	Changite	85	1.	WISHE	-	31,500.0	113	
		64		Ta		W12x16	Knife	144.3438	14.5	100	W39x1S0	Header	Secont?	215		W30499	Header	No Infernation Available	4	
+		2			+	WHENTED	Krafe	Lénéral S	552	2.03	9236150	Header	215/09/21	308	+	9022484		14c0c38	31	
				,	,	W12x35	Co.6s	\$4c3c35	9.5	620	W20x173	Header	United 2	52		WIBATES	Header	(4c3x1/2	*	
46146130	+					Wi2kdS	Krite.	(40)(3)	88	650	W24676	Header	1306316	11.5	٠	W21650	+	130478	11.5	•
- Constant	+					1012235	Kelke	1442438	8.5	0.53	9C4536	Header	DOM:016	ut i		W23450	+	315000	11.5	
Wheelet	-	9				W12-35	Kesfe	1403030	972	0.50	W24x76	Header	0.0500	11.5	٠	W2160		13/0/6/16	511	
	-					WY223S	Knik	(4000)	115	650	W24s76	Header	0.004016	111.5		W23450	-	20000	118	
Manage		34				W12d5	Kriste	14/3/9	165	850	W24s76	Header	0.004916	115	4	W2160	Header	13/0/976	11.5	
0.1100	H			7	-	W12225	Knife	\$4c3c30	115	150	W24076	Header	D-04/216	311	,	W21450	Heady	Dougte	31.5	Ĺ
- 277.00	L	2		7		Wizos	Keefe	(Activity)	155	0.50	W24x76	Header	Hanghe	11.5	÷	W2150	Header	312000	.115	L
W14e211	ŀ					W12235	Sole	16/3/33	38	650	W2606	Header	13/26/16	115		W2150	Header	Dongte	11.5	L
ŀ	ŀ					W12x35	Krafe	LACHOUSE	92	650	W24/76	Header	1304516	HE	,	W2160	F	91/90/01	11.5	L
W146257	H					W12235	Kelle	14-3-19	8.5	650	WZ4vZ6	Header	DOSE	115	,	W2115G	H	31,5600,	115	L
<u> </u>	H					Wt2x3f.	Keife	24/2/2/8	319	650	-W24×74	Header	DLP-SALTE	311	-4	W2150	H	STROKE	11.5	L
W14c283	H					W12236	Keste	(40,018)	9.0	65.0	WQ4v36	Header	Chengis	11.6	,	W21450	H	STREET	11.6	L
H	H			١,		WESOS	Krite	[4e7c28]	992	620	WZ4r76	Header	No information Available			W25450	H	3124513	11.5	L
WMOTH	ŀ			-	-	W12235	Knife	L4e2e3/8	6.5	620	W24x36.	Header	Na InSornation Available			102150	H	100000	315	L
	-					WINGS	Knife	14-7-15	45	0.58	W24x76.	Header	Ne Information Available			W23450	H	305000	11.5	
W14k342	H					WY22235	Kerfe	\$50000 P	972	650	WZ462E	Header	Ne Information Available		٠	W2145.0	H	13/04/16	11.6	L
H	H			-	,	Winds	Suite Suite	(600)	85	650	Wilke?	Header	315/047	11.5	,	W2550	H	130936	115	L
0.146,170	L			-	-	W12235	Krafe	(4434))	99	850	W24s76	Header	D05/16	115		W2350	Header	13/04/16	115	L
-	ŀ			-	-	W13/05	Knife	[46363/B	115	650	3095M	Header	DAMENTE	116	4	W21450	Header	DOSTE	311.5	L
					- 12	W12235	Cuite	14/3/35	185	650	W24076	Header	DOME	115	6	W21450	Header	30,000	11.5	
2000000						W1205	Koife	24/3/3/9	45	0.59	W2606	Header	13/5/07	11.5	÷	W2160		13/0/07/6	11.5	
						W12235	Keste	1403038	92	0.59	. W24v76	Header	31/5/017	115	Þ	W2160	+	0.0476	11.5	
W14c455	+	9				Wrbast	Krafe	(44)438	17	0.53	WITHBA	Header	13x34/16	345	ť	1024455	+	(3/ordy)	11.6	4
•	+					W1223%	Krife	140038	8.5	650	W27584	Header	Charlette	14.5	4	W2465	+	LBrangT6	11.5	1
Whetoo	-				+	WISH	Colfe	\$6000E	912	650	WTBKTOG	Header	Changlie	118		W24455	+	Dongte	11.5	4
2000		na l		i i		W12225	Knife	24x3x3/8	92	0.59	W24x76	Header	Dob-916	511		W2150	+	Charlette	313	-
W14x550	+			ī	T	Wthats	Knfe	(4c2c)(II	52	650	W24x76	Header	13/2×2/16	11.5	÷	W21-50	+	0.0500	115	_
	+					W122235	No.	14223	18	0.53	WERNE	Header	Chchestra.	11.5		82150	+	St. Date St. St.	11.5	1
WHESTO	+	Si Si		14		WESS	KNY	(4c3c)(8	8.5	650	WQ4016	Header	Dx3x5/16	20	14	W2160	+	13-0-67-6	11.5	4
	+					W12235	Corte	260,000	115	650	W24/26	Header	Ukde916	11.5		W2350	-	LPOSÉTIÉ	115	4
WINNESS	+	1	1			Wrads	Koste	(44.343)	99	800	W24s76	Header	0.0940.0	911		W2160	+	Usodys	115	4
-	+	-				W12235.	Knife	[40,038	8.5	0.59	W24674.	Header	1506916	11.5	٠	W21650	-	DOME	11.5	
W14x605	+					WI2dS	Cuite	(4e)(i)	8.5	0.59	W24x76	Header	DOMEST	11.5		W21450	-	130616	113	4
	+	2				W12z35	Kos Ne	E(4/3/2)E	58	0.59	W24576	Header	13/28/16	11.5		W2160	-	Diangle	11.5	4
Without	+	**				Wrads	Colfe	(40)(3))	99	050	W24s76	Header	0.05/0.16	511	+5	W23450	-	1104011	911	4
+	+					W1223E	Keife	(4c)c(3)	117	650	W24076	Header	1349476	11.6	٠	902,545.0	-	1304(14)	116	4
W14c05						W12x35	Keife	TANGE	3.5	650	W24x74.	Header	13/3/6/16	511	4	W21650	-	Lhoight	211.5	_
	4			7		W12235	Knite	\$4c3c338	92	0.50	W24x76	Header	UdeQ16	115	÷	W21150	-	100/016	118	1
W14c210 W16c26		Enite L4chill		8.5	0.59	92450	Trans One		,	y.	W38c300	Header	77 th 628	5%	,	WIGHTS	Healtr	PLR Scott	38.5	
					-	W24x76	£	F14,25x3/2	15.55	:Te	W30c210	Trutt Che	*	6	٠	7621-50	-	915000	34.5	
00.00.00	_	90			£	W24e76	Teats One		ı.		W26/260	Thota Ger	×	Ε	8.	W24484		Undertic	18.5	
	_	G G		10	-	W24x76	ž	F14.25x3/2	\$	or.	W39/260	Trusts Cox	197	9		W2264	Header	13/0/5/16	202	
Existing Col		*		1	4	973478	£	Pittet	345			Truss Che	6)	+3	,	Existing	i.		÷	*
				-	,		4		,		7	*	4				5	14	4	
the factor listed	for each	knife connection	corresponds to	ration	denth to	the 14.5",	deen conne.	Scale factor listed for each knife connection corresponds to ratio of depth to the 145" deep connection analyzed by Ingraffea for axial capacity and stiffness	noraffea fo.	rovial car	Sond things	chiffmann								

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	-		North					South					East					West		
Column	n Stein	- Inpe	55	(mpth [in]	Factor	Bram	Type	ž	Limpth Drill	Factor?	Bean	Type	5.	[u] qubus .	Factor?	Bram	fype	友	[u] qubus	Factor
Witedio			LACACITE	545	+	W24m62	- Sub-	(fechall)	17.5	121	WXXxXX	Header	13+24-216	20.5		WXXXXXX	folk	[Ar.hr.278	215	
1	W36/50	Code	(4x6x3/8)	29.2	2.03	Wibetta	Code	166639	265	1.63	WORLDA	Header	03/04/016	302	108	W24s76	Healtr	0.00%016	11.5	
			E4x3x3B	9.8	150	W12x19	Coh	Leviciti	85	850	W22484	Header	Cakhegrie	14.5	- 65	WIBHO	Header	Dodts	115	
Which	-		54×3×3/8	8.5	650	W12x19	Colf	LACKER	98	0.59	W21457	Header	03/99/20	115	a	W15940	Heider	13/24/216	115	
	+	+	(4x3x3/8)	8.5	650	W12c19	Cally	Lechilli	8.5	0.59	W21457	Header	Chronefito	115	æ	W18s40	Heater	13×5×516	112	*
WStebs		-	24/3/3/3	8.5	450	Wilkits	Cale	16/3/38	95	0.59	W21457	Header	DOM: N	118	-	Williamo	Header	DOSTR	115	
1000	+	+	(4x3x1/8)	9.6	650	W12s19	Kn5	(4c2c3))	8.5	6.56	W2167	Header	Designe.	115	e	W18940	Header	Dodgie	511	*
W14d9	_	-	24c3c30	9.6	0.59	W12x19	Krafe	(44243)	4.5	0.59	W2167	Header	91/94010	11.5	÷	WTBMO	Header	13/0/6/16	11.5	*
		-	LANDAINS	8.5	0.58	W12x19	Code	Lechillis	8.5	929	W2167	Header	21,540,12	118		WYSHAD	Healer	0.04216	112	'
W14x108	-	-	L4x3x3/8	3 8 2	650	Wt2rt9	699	[4c3x]  0	8.5	0.58	W21457	Header	Ukbey16	115	3.	W18s40	Healer	D/D/4/16	1115	ं
		-	(fericity)	8.5	0.59	W12x19	Cish	(400)	97	0.59	W2167	Header	31,040,10	11.5	*	WIB40	Header	thought.	115	
Wikelon	Wibds	-	UNDER	9.5	650	Wt2rt9	Colle	160038	982	650	W21457	Header	0.000.0	111.5		W1840	Heador	000416	115	
	W1205	4	(Acyc)(I)	9.6	650	W12c19	Coule	(40.00)))	98	650	W21557	Header	UNDAG16	11.5		W18e40	Header	13/3/4/16	11.5	.1.
W146130			14x3x3/8	E.S.	650	W12x19	Code	Leadle	115	0.59	W21457	Header	Na Information Available			W1840	Header	Dargite	11.5	*
	W12/05	Knife	16/2/G/B	919	650	W12x19	Dafe	00000	8.5	65.0	W21557	Header	No beformation Available			W1840	Header	Didugite	115	3
(0)16-5117	-	Crafe	Uk/2/cJ/B	9.0	050	W12x19	Coste	Unch/30	8.5	050	W2157	Heater	No Information Available	æ	36	W18H0	Header	0.04516	115	*
	-		LACION	8.5	650	W12x19	Code	Lechalli	8.5	650	W21x57	Header	No Information Assabling	2		W18HD	Healer	Lhoigts	111.5	
25 ×25 ×25		_	14×2×3/8	98	650	W12x19	Colfe	16/3/38	8.5	0.59	W21457	Header	Us945/16	111.5	*	WTBMO	Header	13/3/6/16	115	
	W12k35	Sult.	(4x3x3)	9.6	650	Wt2st9	699	16000	8.5	950	W21157	Header	Landerin	115	- 27	WIBHO	Healer	Dongte	115	
101340-502	W12x35	Krite	14×3×3/B	8.5	650	W12x19	fort	Lechtiff	98	0.59	W21s57	Header	L3x3x5/16	115	,	WIBNO	Header	UND4916	115	
	W12k35		L4x3x3/8	8.5	0.58	Wt2xt9	Code	160338	50	0.58	W21s57	Header	L3x3x576	115		W18e40	Header	DAMETE	11.5	*
thrash colo	2002198	Code	144.3435	8.5	450	Wthth	Code	16003	85	950	W2157	Header	DOMESTIC	115		W18HO	Healtr	Livorgite	115	*
	W12x35	KNA	24x3x1/8	3.6	450	Wt2s19	Keise	14-3-33	8.5	0.59	W2157	Header	1349451E	115	100	W18940	Heafer	13/24/16	111.5	
100.00.003	W12x35	Chille	L4x3x3/III	4.6	62.0	W12x19	Crafe	14×2×30	45	0.59	W2167	Header	13/3/6/16	211	- 60	WIBNO	Header	13/29/210	11.5	.2.
			L4x2x3/5	40	650	Wibits	Code	Lecholik	58	0.58	W21457	Header	13/24/216	11.5	-	WISHO	Heater	Dodgte	118	1
W14x176		-	L4x3x3/B	8.5	62.0	Wt2x19	Code	Lechild	8.5	62.0	W21657	Header	13/29/216	11.5	æ	W21544	Header	LNON916	11.5	ं
		-	(4x3x378)	8.5	650	W12x19	Cafe	56309	972	0.58	W2352	Header	0.05676	11.5	+1	W21544	Heafer	Chokys	11.5	*
W14x176		+	LACACITE	8.6	650	Wthtt	Cash.	(40,03)	9.6	0.59	WC1857	Header	13/340/16	11.5		W21s44	Header	DOM'N	211	1
	W12/35	+	(4x3x3/8	20	850	Wt2ct9	Costs	1603/38	88	650	W21557	Header	Choogse	115	٠	WISHO	Heater	Chabelle	113	*
W14x133	1	+	(4c2c3f)	4	650	Wileta	Cont.	(entitle	82	0.50	W23457	Header	11,000,10	11.5		Willeto	Header	Changing	112	1
	WLEGS.	Cak	(actain)	480	0.00	W12519	Ann.	160,000	88	0.58	W2167	Heater	13-0-476	200		WIBMO	Health	Ukangilis Ukangilis	118	
W16c193	_	+	through	8 80	0.00	Wilsell	Contraction	14/3/30	98	0.00	1071467	Hander	DANGE I	2 1		William	Heater	Living 16	418	1
	t	H	(4c3c3/8)	88	650	W12x19	Code	(60.03)	8.6	0.58	W2167	Header	13/24/216	11.5	3172	W1940	Heady	Doğum	115	-
W146211	+	H	16/3/3/8	98	0.59	Wibit	Code	Lectoria	8.6	0.59	W21457	Header	0.09636	11.5		WIBNO	Header	Changing	115	1
Mary No.	Wibas	Colk	LACASTR	9.8	650	W12x19	Colt	(40,03))	9.0	0.59	Wütsez	Header	13×3×676	11.5		WIBNO	Header	0.00476	415	1
73044	W1205	Code	14000	9.0	62.0	W12x19	Chilly	16050	9.0	0.69	W21s57	Header	Chokers	115	20	W1840	Header	131/29/210	11.5	2
90140333			1,44,34,378	38	650	W12c19	Code	1600(8)	9.0	650	W21x57	Header	13/5/6/16	11.5		Withsto	Healer	DOMESTIC	11.5	
		Knife	(4c2c1/8	8.6	620	W12x19	Knife	(60.03))	38	0.50	W21s67	Header	Dobyte	115		W18s40	Healty	Didugite	11.5	
Withous	_		L4x2x3/III	4.6	650	W12x19	Knith.	16(3(3))	4.5	0.59	W2167	Header	13/3/6/16	11.5	(4)	WTBHO	Header	13/0/616	11.5	*
		-	Ukracitii	58	650	W12x19	Code	(600)	98	0.59	W21457	Header	UNDAG16	111.5		W1840	Heater	Chongre	115	
W145-00E	W24c76	Truss Che			13	W24c76	Truss Cos		+	æ	MG-28	Header	13x34516	3.5	35	W21550	Header	13434216	118	
	W240'6	£	F14.25x172	15.25	+3	W24s76	£.	Pt 4.25x U2	21.25	+	W24n68	Header	Dich076	10,5	*	WC1650	Healer	0109610	11.5	*
W14x730	-	, i	(4)	. 9	9	9/24/26	Truss Ches				W39/100	Header	1349451	28.5		Watero	Header	DOMEN	11.5	
	+	+	84,25492	15.25	×	W24-76	£	PI 4.25x U2	£	œ	Whr31	Header	Us/245/16	98	÷	W24x62	Header	Diodite	115	å
W34x730		E.	+	12	0	W24x76	Inns Om				M5-31	Header	13434516	2115		W2465	Header	0304016	10.5	*
	W24c76	Œ	P14.25s.122	-	1	W24s76	ď	E4 2 740, 164	4		1400 0 0 0	77	13000000			M251m03	Strafer	140500140		

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Without   With	1909   1919	7 K th / 1 K	eag	<u> </u>	£ 42 62 x 62 x 62	Length Gri	Factor!	Beam	Type	5.	[within]	Factor?	Even	fype	5	Length Inl	-
Withties   Gode		400 a			62 63 2 63 8 9 3 1		11	W24stl3							-		
W2041   W2040   W2041   W2041   W2041   Gode   W2		# # # # # # # # # # # # # # # # # # #			2 6 2 2 60 2 8 8 3			1	Header	Lbchs716	115		W21544	Header	Lb04516	115	
1972-19   10-04     1972					6912 69 8938	÷	æ	W23-84	Header	0.69676	570	æ	W24s76	Header	UND-5716	17.5	
WT9.251   Gode					2 9 62 60 8	4:		W22k130	Header	CANASTE	205		W24M62	Header	Dodte	30.5	
WT9231   GOOK		## 1	X 52 5 53 57 8 55 8 7 8 5	× · · · · · · · · · · · · · · · · · · ·	2. 439 36 P. 26 3	9		Wotes	Header	UN98/10	11.5		W1840	Header	13x3x5/16	98	
WESSES   GOOR		800 800 800 800 800 800 800 800 800 800	5.02 6.23.05 862 8.50.0 8.24.5 8.2		50 5 P.O. J	ж	æ	W21x68	Header	CN29-576	115	æ	W18HO	Header	0.05616	9.0	
1972-93   Gode		# # # # # # # # # # # # # # # # # # #	2 8 2 8 8 7 8 8 8 8 7 8 8 7	2 8 2 2 8 2 2 2 1 1 2 1 2 1 2 1 2 1 2 1	2 3 10 2 3	ti		W21x58	Header	DOSH	115	-	Whilesto	Header	DOST	8.5	
WT9231   Gode     WT9231     WT9231     WT9231     WT9231     WT9231     WT9231		##0 ##0 ##0 ##0 ##0 ##0 ##0 ##0 ##0 ##0	X 2. X 3. X 2. X 3. X 3. X 4. X		35 P. 25 B	ž	12	W21x68	Header	LIVES/16	115		W18540	Header	Dobblie	88	
W19291   Gole		850 850 850 850 850 850 850 850 850 850	63 83 8 58 8 6 6 6 6 7		D. C. C. C.	÷	X	W21x60	Header	0.04010	11.5	÷	W15s40	Header	91590011	8.5	
100,000   10,004		## 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	8 80 8 58 8 48 50	2		e.		W2168	Header	DOMES	115	10	W18640	Heador	0.04916	85	
100,00.00		850 850 850 850 850 850 850 850 850 850	812 8 SIS S 315 A	N 2 2 5 5 8 5 5 5 7		æ	38	W21x68	Header	3124040	115	35	W18s40	Header	13x34416	35	
90.9249   Gode		850 850 850 850 850 850 850 850 850 850	0.8 55 5 55 50			÷	Y	W21s68	Header	DOMESTIC	11.5		WIBNO	Header	11/2/0/11	115	
1970-19   10-04     1970		650 650 650 650 650 650 650 650 650	x 6x x x x x x x x	2. 5 5 8 5 5 5 7		4		W21x68	Header	Uds916	111.5		W1840	Header	DOSTE	9.2	
1979-191   Golde     1979-19		850 850 850 850 850	535 X 144 143	565 8 5 5 5 7	(8)	9	3	W2 tx68	Header	United/16	115	*	W18H0	Header	3326216	8.5	
WODD   GORE		850 850 850 850	X X 1.4 ( )		89			W21x68	Header	0.04916	11.5		W1840	Heater	DOME	8.8	
90223 GOA- 90224 GOA- 90225 GOA-		850 850 850 850 850 850 850	E 345 60		×	*	1.5	W21sea	Header	DANG16	11.5	3	WIBNO	Header	Lhoighte	912	
902-20   Gode		850 850 850 850 850 850 850			*	×	*	W21x68	Header	31,500.0	11.5	36	W1840	Header	0.00516	85	
WO255   GORE		850 850 850 850				Ŀ		W21x68	Header	13:04576	118		WTB40	Header	Litoritis.	85	
WINDS   GOAR		850 850 850 850				+		W21v68	Header	13/2/01	1115	*	WTBMO	Header	13/3/6/15	8.5	
10,000   Gode		650 650 650						W21458	Header	Lavakýté	4115	67	W1840	Header	Dougte	9.5	
W0231   Gode		450 450			9	4		W21s68	Header	Dcb5/16	115		W15e40	Header	13/04/216	8.5	
979251 Gode		450		ě	٠	*		W21x68	Header	LINDAS/16	115	٠	W18e40	Header	LNOSTIE	8.5	
100.00   1		850				÷		W21459	Header	DAST	11.5		Willedo	Header	Lhospie	8.8	
100 to the   100		650	2	2	0	Œ	2	W21x68	Header	31/54/AE1	115		W18840	Header	13/5/4/10	88	
10,000   1		2000	8	25	(20)	ě		W21x68	Header	0.000.00	311		WTENCO	Header	1304216	4.5	
100.00   10.		650	,			-	,	W18x71	Header	Linds 16	11.5		WIBNO	Header	Chchig16	18	
W7249 Gode		850		e	[6]	ж	*	W21668	Heade	13x3x5116	11.5	*	W21544	Header	LNONS116	113	
(004 (004 (004 (004 (004 (004 (004 (004		650	10	* 1	*1	10	10	WZTHEB	Header	C3x2x57.6	11.5	*	W21544	Header	51,500,010	11.5	
W1225 Gok W1245 Gok W1245 Gok W1245 Gok W1245 Gok W1245 Gok		650						Wohells	Header	1349434	11.5		W21584	Header	1309616	211	
W1245 Gold. W1245 Gold. W1245 Gold. W1245 Gold. W1245 Gold.		250	+	,	*			W21958	Header	CRASTE	511		WISHO	Header	CRSETIE	82	
W1243 Gale W2249 Gale W2249 Gale W2249 Gale		650		9				WZDeta	Header	DOSM	112		WIB40	Header	Change	12	
W2x19 Knik W2x19 Knik W2x19 Knik		0.00	,		+ 0	+ 3		WZDSS	Header	312400	100	,	WIBNE	Header	Changes	88	
WINESS Keile Wilkery Keile	140,000	0.00						WING	Nessee .	1349636	100		Wilder	Heater	130,6216	200	
Witota Golfe		0.00		0.0				MONTH	- Treater	13/0/61/6	344	312	Order	No. of Street,	1304616		
		0.00						WC1468	Header	DANGE	98		William	Header	thought.	34	
W12x19 Code		650						WOTHER	Header	D-09/16	115		WIBNO	Header	1304216	99	
Confe		650	28.	2	300	9.	æ	W2 tx68	Header	0.000,10	311	25	W10eto	Header	Lindson	9.9	
Writeria Coults	Lecholfs 8.5	450						W21x68	Header	13424516	11.5		Withful	Header	13/24/16	818	
White White Cafe Car	(4c)c(1)) (1)	620	2	0		æ	22	W21s68	Header	DOM:N	511		W18640	Header	DAD4716	9.8	
W12x19 Knife UAx	Lechilli at	450	93	8		*	90	W21x68	Header	01/540401	115	100	WTBMO	Header	13/0/616	145	
Wildell Knitt	Uncode 85	650					,	W21458	Header	Dob\$16	115		W1640	Heater	Chongre	52	
Truss One	+	35	W16c26	Colt	\$4ch30	982	0.59	W36:210	Heider	8.5458	π.	35	W18465	Header	D-04576	118	
W24c76 hm	M 4.25x 15.2	+	9627844	£	FI & 73x U.2	2		W30x210	Truss Con		* 1		William	Header	01596510	68	
	The same							W395300	Truss Cas	* 7			Wideritt	Forth.	144243	8	
117400						,		Will don	and and	ę			Wd troop	100	144.00.00		
1001001			6	9				Water	India Con	0			West	200	144.243	2	
W24476 Fm.	78 7 End # 78	+	+	*		÷	*	W36/300	Insta Cox	*	+		W18-40	Kna Pr	1442429	11.5	

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COLUMN 67	
CATALOGUE -	
ONNECTION TYPE	
INTERIOR COLUMN C	
.10	

	-		North					- South					Elet					West		
Column	Seam	- Inpe	差	length [re]	Factor?	Bram	Fype	5.	Limphifing	Factor*	Bean	Type	5.	[within]	Factor?	Beam	fype	204	[w] qubury	Factor
Wited®	10	6	8	6	6	903465	Code	CACALIER	6.5	650	W30x116	Header	13x3x516	14.5	53	W/95c150	Heador	Lbosg16	312	55
WILESTO	2		100	94	×	W36x150	Code	1,616.30	29.5	2.03	W26x176	Header	03/04/016	345	2	W36c150	Header	13:04576	20.5	:
1000	0	4.7	-			W10-26	Keh.	(fe/Je/Ji)	85	959	W30k173	Header	SACHIZ.	×	- 62	W264230	Header	14e3cl2	215	•
1114-111	9	9		91	÷	W1845	Colfe	Lechilli	14.5	1.00	W30x132	Header	Uxb4276	205		W24-76	Header	13x3x5716	11.5	1
	*	35	36	1		W14c22	Cristy	L4C3-339	8.5	0.53	W23484	Header	DX345/10	345	æ	W24s76	Header	01/54540	115	*
W14x158	•	6	ti)	4.5	40	W14c22	Code	16/3/38	9.5	650	WZX346	Header	DOSH	17.5	-	Waterle	Header	Does	115	**
G Section 1	2	9	ä	9	2	W14622	Knife	24×3×30	8.5	0.68	W22484	Header	DAMPE .	14.5	0	W24s76	Header	UNDERTE	11.5	2
W146-193	£	*	(6)	ť	20	W14s22	Code	(44.35.30)	45	0.59	W27x346	Header	01/5/07(1)	17.5	*	W24s76	Header	13/04/216	11.5	15
21/20/21/2	0.	D.	(F)	,	,	W14c22	Code	14c3c33	88	929	W27484	Header	3154043	14.5	25	Waters	Heador	0.09516	115	2.
10.07	31		3.5	9.	28	W14x22	Kn3k	14<0.30	8.6	0.50	W27x146	Heider	13/24/216	17.5	35	W24s76	Header	Lb/b/g/16	115	8.
	ε	8	*6	×	30	W14422	Cish	00000	8.5	950	W27484	Header	0.09/016	14.5	35	W24c76	Header	DOGUE	11.5	
the count	7	7	187	4	,	W36/280	Seated		-		W36/170	Header	05/07/0	385		W24676	Header	0.000	115	7
1070010		3	3		93	W14s26	Coult	(4c)c)(8	98	650	W24s76	Header	UNING16	11.5		W24s76	Header	3129600	11.5	đ
- meaning			*		0	W14x26	Code	164338	115	450	W24r146	Header	Uchilli	14.5	-0	W24s76	Heater	DOMEN	11.5	*
W1482111		3			×	W14k26	Klafe.	Decision	8.8	950	W24x76	Header	DA95/16	11.5	:	Worker	Header	Didegle	115	
444-444	*	c	*		×	W14s26	Cooke	Unchills	8.5	050	W24x146	Header	00000	145	36	W24s76	Header	Na Information Available	×	15.5
74774714	0	0	**	1.0		W14c26	Crafe	(4e/p/10)	8.5	650	W24s26	Header	13434516	115		W24-76	Heador	Na Information Available		*
ORCHAUS.						W16/26	Crite	(4chille	8.5	0.59	W24x146	Header	05/0/03	17.5	*	W24s76	Heater	No Information Available		
0.00				7		Wzbettill	fish	(feet))	365	1.83	W2244	Header	DAMETE	14.5	- 65	W24A76	Header	Na Information Available		
00144-300	9	9		,	,	W14x26	Knitt	technilli	982	0.59	W24x146	Header	12-6-38	145	,	W18-97	Header	DAMS716	115	1
646764		*	(0)			W14x26	Crete	Lechalls	200	0.58	WZ4xZ6	Header	LININGTE	115		WZ4kJE	Header	Lixpigite	11.5	
MYANATH	•		*:	4-1		W14x26	Colt	16:3:28	9.5	950	W24x346	Header	Chdicis	345		W24s76	Header	Dogte	115	53
	0		er.	20	0	W14x26	Colfe	24ch 398	8.5	0.59	W24s76	Header	13434576	115	2.5	W24s76	Header	Changite	511	
WHANEE	20	Œ	*	£	E	W24x26	Krafe.	145.25.33	3.5	0.59	W24x346	Header	C3×2×3/8	14.5	(4)	W24x76	Header	13/3/6/16	11.5	ž.
The same of	-	ř.	23			9714426	- toy	(4c)citi	58	-0.58	W24x76	Header	13434516	115	15	9245A	Header	Upda6716	115	1
00146500		ě	œ	8	÷	W14k26	Crafe	Like Stride	8.5	62.0	W27x346	Header	Carange	17.5	æ	W27x84	Header	DOMENTO.	14.5	*
	+1	ŧ	ti	5	*1	W14426	Code	56/3/39	8.5	0.59	W27484	Header	03/59/50	14.5	*	W27584	Header	0.09600	14.5	*
W14x550						W14425	Ent.	140,033	9.5	0.59	W34x346	Header	Lishing	5.0		Without	Header	1309616	115	
	•	+	*	,	•	W14626	CONTR	Lectrific	9 1	650	W24876	Header	CINDAGUE	511	٠	W24876	Header	Chronical	11.5	*
W146605			*:	es		WHERE	200	Certain Co.	82	020	W26c146	Header	Debugs	2 2		Weers	Heado	Character	113	572
0.0000000			9			Wideou	Total Contract	Tay by 10	0.00	0.68	Whitehall	Header	(363638)	27		Works	Header	Dodge	311	
W14x605	ŀ					W14/26	Code	CACALIER	8.6	650	W24s26	Header	03/05/06	11.5		W24c76	Header	DOSTS	115	1
	9	9.		. 1		W14x26	Code	(4c2x3)	8.5	0.58	W26x146	Header	05000	316	2	W24676	Heador	DOMESTIC	11.5	0.
W148565	+	+	*	*		W14426	Crafe	16/3/39	88	0.69	W24x76	Header	0.00000	11.5	*	W24s76	Header	thospie.	115	
WALLES		9	35		,	W14426	Code	Lechan	9.6	0.59	W24x346	Header	UNDANS	14.5	,	WARTE	Header	0304616	115	
		×	96	2	20	W16/26	5995	140-30	8.5	0.69	W24s76	Header	Changite	115	20	W24s76	Header	13/2/6/10	1115	*
W14x730	2.	÷		,		W14e26	Code	(4c2c3)	9.0	0.09	W24x346	Header	UADA98	14.5		W24s76	Heador	Changing	11.5	
	2		Œ.	,	0	W14s26	Knite.	((4/24))	88	0.29	W24s76	Header	DANGH	115	2	W24x76	Header	Lhchegris	511	12
W18x730	£	4)	8		e	W14s26	Krafe	(40.00)	98	0.50	W24x346	Header	(3003)	14.5	6	W24c76	Header	13/04/216	11.5	
		+				W14c26	tok	(40,000)	98	0.59	W24e76	Header	13/09/216	11.5		W24c76	Header	Chongre	11.5	1
W14x730	W1626	Cult	(4c3c3/l)	3.5	0.59	W24c76	Trust Ow		+	35	W36/300	Header R	8754614	g.		W36x300	Header RI	Ft 9x5/8	223	3.
	ti	ŧ	10	1	+ .	W24s76	£	F14.25x1/2	15.25	1	W30-210	Truss Cro	*10	ri	r.	W26x210	Trutts Ons	85	+	*/
W14x730	9					W24x76	Trust Cos.	9)	(4)		W39:300	Truss Cas				W39-260	Truss Ons			1
		88	(ir	9	ķ	W24-76	£	Pt 25x1/2	£	98	W364280	Truss Cres	36	<b>.</b>	::	W392360	Trusts Chis		38	d.
Sits on excelling Col	Col	6	Ť	10	-	W24x76	£	Pitts	74.5	9		Truss Ora	60	42	*	*	Trans Cra	9	9	83

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity
Where no beams size is indicated no member is framing into the column
The Information specified for each connection was taken from the latest corresponding frankel Steel Limited steel shop drawing for that connection

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Code         Choice         Choice <th>Learning Tool</th> <th></th> <th>-</th> <th>Si .</th> <th>South Core</th> <th>Const</th> <th>Page 1</th> <th>- Lone</th> <th>18 5</th> <th>Lanceth Lan</th> <th>T. Cont.</th> <th>The state of</th> <th></th> <th>3</th> <th>Period Period</th>	Learning Tool		-	Si .	South Core	Const	Page 1	- Lone	18 5	Lanceth Lan	T. Cont.	The state of		3	Period Period
4.4.0.         1.4.5.<	+		-			-		+	Delegate 16	311		3	1	ofer 1	5
WINDS         CHA         CHADAD         CHA <th< td=""><td>-</td><td></td><td>-</td><td></td><td></td><td>H</td><td></td><td>H</td><td>01/946/10</td><td>245</td><td></td><td>Wijktis</td><td>F</td><td>de</td><td></td></th<>	-		-			H		H	01/946/10	245		Wijktis	F	de	
MINDEZ         CORA         CHADADIA         61.0         TOTALITY         CHADADIA         61.0         CHADADIA         61.0<							Н	Н	13x3x5716	14.5	5	W27x84	H	Header	
Method         Conday         18.5         0.00         Method         18.			1					-	UNDAS/16	205	*	+	+	ale de	
(1) (1) (1) (1) (1) (1) (1) (2) (2) (2) (2) (2) (2) (2) (2) (2) (2		+	1				1	-	Choograg	118	20	+	+	į.	-
Mind Street         Color of the color	-		+			+	$^{+}$	+	10000	2 2		WORSE	Health Co.	1 1	der (100578)
WM 202         Cheb         Lub 2010         61 Ce 10 Ce 1			+			-	H	+	130476	7.	*	t	+	1 2	
WMACKS         Cock         COCK STATE	-	H	H				H	H	Dodgie	115	ľ	H	H	,	L
WMA20         Color         Color         Color         WMA20         Color         WMA20         Color         WMA20         Color         WMA20         Color         WMA20         Color         COLOR         WMA20         Color         COLOR <th< td=""><td></td><td>H</td><td></td><td></td><td></td><td></td><td></td><td></td><td>UNDAG16</td><td>57.</td><td>25.</td><td>W21467</td><td>S7 Headyr</td><td></td><td>r UND4216</td></th<>		H							UNDAG16	57.	25.	W21467	S7 Headyr		r UND4216
WHANGO         CARCA         CARCAA         CARCA         CARCA         <		Н							DOME	311		W21x52			
WHACE         COLAR         COLAR <th< td=""><td></td><td>Wt</td><td></td><td></td><td></td><td></td><td></td><td></td><td>01/04/010</td><td>28.5</td><td></td><td>W21657</td><td>S7 Header</td><td></td><td>, Lbdx916</td></th<>		Wt							01/04/010	28.5		W21657	S7 Header		, Lbdx916
WEAT OF CASE (A.C. IN CASE)         CASE (A.C. IN CASE)         TIAN TO SECURATION OF THE CASE (A.C. IN CASE )         CASE (A.C. IN CASE )         TIAN TO SECURATION OF THE CASE OF THE C	2		-						01/2×6/16	11.5	*	+	-		-
PRINCIA         COMPA         CARDAD         CARDAD<	-	+	+			+	+	+	Dogs	100	1	+	+	- 1	Ne leformation Available
WH MACH SI CARDA         CARDA SI CARDA         CARDA SI CARDA         CARDA SI		+	-			+	+	+	Charleg 16	113	1	+	1	- 1	No Information Available
NYMEY         CARD         CARD <t< td=""><td>-</td><td>t</td><td>1</td><td></td><td></td><td></td><td>t</td><td>+</td><td>STOPPE I</td><td>2 1</td><td>0</td><td>We total</td><td>The state of</td><td></td><td>No behavior for the fall</td></t<>	-	t	1				t	+	STOPPE I	2 1	0	We total	The state of		No behavior for the fall
WIRD-18         Cohe         Lock-10         31.5         1.10         WIRD-18         1.		t	+			-	t	H	13/24216	302	1	W21457	H		ŀ
WHACKS         Color         CHACATOR         COLOR         CHACATOR         STOCK AND COLOR         CHACATOR		t	-			+	t	H	(3x3x5716	1 1	1	W21452	H		SPACE
WHORES         Cock         LEADING         STACE         LEADING </td <td>ŀ</td> <td>t</td> <td>H</td> <td></td> <td></td> <td></td> <td>t</td> <td>H</td> <td>UNDANE</td> <td>14.5</td> <td>1</td> <td>t</td> <td>H</td> <td></td> <td>0309216</td>	ŀ	t	H				t	H	UNDANE	14.5	1	t	H		0309216
WHACKS         Cocke         CADADRIA         CADADRIA         WEADARD         CADADRIA         CADADRIA         CADADRIA         CADADRIA         CADADRIA         WEADARD         CADADRIA         CA			H				t	-	DASSTE	115	ŀ	H	H		Lingste
WHANG CACHE         CACHE AND CACHE AND THE AND CACHE									CNRSS	14.5		V2157	S7 Healtr		though
WRACK         CACADA         CACADA </td <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td>DANGTE.</td> <td>115</td> <td>2</td> <td></td> <td>S Heafer</td> <td></td> <td>31/540431</td>									DANGTE.	115	2		S Heafer		31/540431
WH 650         Color         LADAGO         TASA         CADAGO         TASA	+	+	-					+	00000	14.5	8	+	+		0.09610
WR4021         Cold.         Cold. <t< td=""><td></td><td>+</td><td>+</td><td></td><td></td><td>1</td><td>1</td><td>+</td><td>13chg16</td><td>11.5</td><td>1</td><td>+</td><td>+</td><td></td><td>DOM:16</td></t<>		+	+			1	1	+	13chg16	11.5	1	+	+		DOM:16
WEAT OF CASE (A.C.)         CASE (A.C.) <td></td> <td></td> <td>+</td> <td></td> <td></td> <td></td> <td>+</td> <td>+</td> <td>(3434)8</td> <td>14.5</td> <td>*</td> <td>+</td> <td>+</td> <td><math>\rightarrow</math></td> <td>13/29/5/16</td>			+				+	+	(3434)8	14.5	*	+	+	$\rightarrow$	13/29/5/16
PRINCIA COMP.         CARDADIS         18.5 CARDADIS         CARDADIS <td></td> <td>+</td> <td>+</td> <td></td> <td></td> <td>+</td> <td>+</td> <td>+</td> <td>Diche216</td> <td>115</td> <td>*</td> <td>W21852</td> <td>+</td> <td>+</td> <td>Chongre</td>		+	+			+	+	+	Diche216	115	*	W21852	+	+	Chongre
WH 6451         Code         Leb 2019         18 C 10 10 10 10 10 10 10 10 10 10 10 10 10			1			-	t	-	Christin	11.5		WORKS	D Header	+	Lizavite Lizavite
Windold         Cook         Cook Day         Windows         Cook         Cook Day         Cook		H					t		Uchilli	2	*	H	H	⊬	Undelgite
WR 645         Cohe         Cohe Bill         Cohe Bill         WR 540         Cohe         Cohe Bill									0.05676	511	17	W2tx57	2 Header	Н	DOM:
WRACK         Cock         Cock         Cock         Cock         TORAGE									(3×2×3)	14.5	-	W21x57	-	-	LbdsQ16
Third   Cork   Colon   Colon								-	13/2/6/16	11.5			-	-	0.00516
Winted   Cock   Location   18.5   Cold   Winted   Cock   Cockin   18.5   Cold   Winted   Location   18.5   Cold   Winted   Cock   Cockin   Cockin			+					-	00000	14.5	2.	W2167	+	-	0.04916
Winted   Cock	1	+	+			+	+	+	0.000016	8.5	-	+	+	+	thaspie
Witted   Cock		+					t		(Depoils	14.5		W2 No.	Meader 1	+	0.000010
WINGO         Color         Unball         STS         COLOR         COLOR         COLOR         STS         COLOR         COLOR         COLOR         STS         COLOR		t	+				+	+	Choballi Choballi	0 2		Wohled	+	+	1303676
Windows         Cohe         Cohe Maria	-		-			-	t	-	DOMESTIC	115		-	H	+	Dodgie
With 64 Cark         Cark         Cark         Use Date         85 GR         Find Date         Find Date<									0.000	14.5	.67	W2167	22 Header	$\vdash$	13/0/6710
Wide All Strates         Fine Column (Wide All Strates)         Fine Column (Wide All			-					-	Dodgte	11.5	-	WZ1HZ2		$\rightarrow$	Changes
Wild by Factor         Teach of Fa	+	+	-			+	+		UND-5/16	14.5		+	+	$\rightarrow$	Undel/16
Wide Model         Francis         19.4 Zeros		+	+				W24w	+	13/3/216	12.5	*	WZews	+	$^{+}$	CHOMETE
18543 har/be 7 1851 har/be 1851 harber 1550 har/be 1851 harber 1550 har		+	+			1	Water	1	13000216	200	9 3	W39x150	No Header	-	Dongte
		W2	+				M5-31	+	13434216	215		+	H		Changing
. W240'S Fee 1972'SGUR 18 - W300'S Heater L3X06'16	.5	WS	L	Fin 1937	25c34 18	*	W3043	13 Header	0.00676	275		W3069	13 Header		315000

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			North					South					East					West		
Column	Seam	- Libe	Sider	length (in)	Factor?	Beam	Type	254	Limpth Drill	Factor?	Bean	Type	5	[mithin]	Factor?	Bram	fype	Sar	[w] quban	Factor
Wited®	W14422	616	L4x2x3/B	8.5	0.59	6	6	8		55	W24s62	Heider	L3x3x576	115	53	W24n58	Header	Lhchg76	115	5
Widelin	W24455	Park.	(4x3x3/8)	17.5	121	9.	e.	24.		×	W24s62	Header	03/34/216	14.5	æ	W23/54	Header	L3x3x5716	17.5	3
	W12k19	Chile	S4x3x3/B	9.6	659		0	6	6		W38KS0	Header	Doneyte	20.5	- 00	W20x130	Header	Changité	20.5	0
101.6-615	W14422	Colfe	UACHUM:	8.5	050	,	91		4		W27x84	Header	UN245/16	14.5		W23x68	Header	13×3×5/16	115	
	W12x19	Cook	14x3x3/8	8.5	650	×	×	26	ж	æ	W24x68	Header	Ux35/16	115	æ	W2 total	Header	01/5/0473	115	*
00.785.000	812519	Code	144.34.378	8.5	650	5	-	- 1	ti		WZ4xSB	Header	Meletemator Available		-	W25x58	Header	Lhoistis	11.5	*
	Wt2x19	KNS	54x3x3/8	9.5	659	2.	2	æ	3	æ	W24s68	Headto	Natehmaton Available	y		W2 ts68	Header	LNONGTE	115	2
10140-111	W12k19	Krish	L4x2k3/lll	9.5	0.59	90	N.	*	÷	80	W24×60	Header	Noteheaton Available	Œ.	*	W21s/(8	Header	13/24/210	11.5	10
2000	W12c19	Code	144.34338	8.5	0.58	,	100	JESS	6.		W24eSB	Header	No latematon Available	0.0	20	W21568	Healer	21,540,012	115	2
1074-108	W12x19	Knik	L4x3x3/B	98	650	es.	28		34	M.	W24x68	Heider	0.000,016	11.5	a.	W21468	Header	L3x3x9/16	115	8.
MINNESS OF THE PERSON NAMED IN COLUMN 1	W12k19	Colfe	24x3x3/l)	8.5	650	8	8		÷	Y.	W24s68.	Header	3129-610	11.5	85	W21568	Header	DOME	11.5	*
100,000	W12x19	Code	CACACITI	98	650				4		W24x68	Header	9,500	115	7.	W21568	Header	DOSTE	115	2
	W12k19	KNA.	1400038	8.5	650	×	×		3	æ	W24s68	Header	03/29/26	11.5	20	W21x68	Header	13x3x6716	115	3.
- Anna	W12c19	Colt	L4x2x378	8.8	650	5	5	E			W24e88	Header	0.04916	11.5		W2 bids	Header	Undelgtik	11.5	5
W148/33	9912419	Krite	S(DCH)	919	650	A	A		٠	i.	WZ4v68	Header	31,540.0	11.5	:	W2 tx68	Header	1306/16	115	3
10000000	WI2019	Chille	UNDER	98	650	x	8		÷		W24e68	Heater	31,54017	311	*	W2 tx68	Header	915900	115	*
W166/33	Wi2et9	Colfe	LACRETTS	8.6	650	,			1.		W24stills	Heider	13:04/216	115		W2 tx68	Header	Lhoist16.	115	
100.000	W14/22	Kish	twarife	988	650						W24476	Header	31/24010	11.5		W2 tx68	Header	L3x3x576	115	
n 143.50	WITHTO	Shift	Unwill	38.5	1.83						WZBCZDG	Header	DOMETE	202		W21468	Header	Dougte	511	
10114 2005	Wikits	Stalk	140,000	99	650	,	,	29	4		W24s62	Header	UN98710	115	,	Woteds	Header	13x3x5/16	415	
100700100	W12e19	CHR	L4x3x3/B	8.5	0.59	8	8	(8)	90		W24s62	Header	LINING/16	115		W21x68	Header	LDONGTR	11.5	×
the state of	812518	Code	144.34.378	88	450		- 0		÷		W24e52	Header	DOST	115		W21x56	Header	Lhoigts	115	*:
	W12x19	KNS	24×2×3/8	200	659	92	22		Œ	92	W24e62	Header	13404516	115	22	W2 tx68	Header	13x3x5/16	41.5	15
(96,000)	W12x19	Knife	14x3x3/III	4.5	0.59	9	90		*	0.0	W24x62	Header	13424570	11.5	(4)	W2 tx68	Header	134245/16	11.5	2.
	Wilking	Colfe	LAx2c378	9.5	629				4	,	-W24M52	Header	13x3x516	11.5		W18x71	Header	1349675	115	1
00000000	W12x19	Code	LikeJeJjë	3.5	620	*	81	Œ.	*	e.	W24x52	Header	Ux245/16	11.5	æ	W2 txGa	Header	13/29/516	115	ð
	W12c19	Cuth	(4x3x3)B	8.5	0.59	85	10	*	6	*	W24s62	Header	030676	0.5	*	W21x68	Header	13/3/6/16	11.5	50
WYANTER	W12x19	Snike	Like he hills	8.5	650		1	*	5		W24x62	Header	13/54/26	11.5	· e	W21s68	Header	13/24/216	11.5	9
	WESSER	Coste.	L4x3x328	8.5	620	E+1		.+	+	٠	W24x62	Header	13x3x576	115		W21x58	Header	13×3×6/16	115	*
W148426	W12c19	Colle	L4x2x3/8	3.0	650		9	*	t		W24e62	Header	13:3:516	11.5		W2 tx68	Header	Usangne	11	5
	W12c19	Colfe	(4x2x1/8)	3.0	650	÷	÷	12	3	?	W24e62	Header	DOMEN	311	7	W2 trés	Header	UNDATH	311	*
W14x455	WINE	KNA	14020378	382	650	×	8				W24v62	Header	13/39/216	115	-	W18/71	Header	Lindague	11.5	*
	WINE	SN.	CACTORIN	28	500		0				W2446.2	Header	DOM: 10	11.5		W2 bysit	Hesdey	Likhig 16.	11.5	•
W146500	W12419	Costs	[4c3c3/8]	9.6	650	2	2	2			W24s62	Header	13/3/6/16	11.5	2.	W2 tx68	Header	0.00/676	118	2
	W12419	Colfe	LACACAB	8.6	650						MG-142	Header	LhdsQ16	8.5		M0-142	Header	1004916	88	1
WYANGOO	W12x19	Stalk	14×2×3/8	8.5	650			9	9		W24x62	Header	13/39/210	115		W21568.	Header	13/04/216	11.5	1
	W12c19	Code	14000	9.5	650	ž	×		31	æ	W24x62	Haaler	03/99/10	11.5	æ	W2 to 68	Header	13/2/6/10	115	2
W14e550	WYZYTB	Code	(4c)c)th	3.6	650		-	20			W24n62	Header	13434516	11.5		Worklings	Header	13/04/16	11.5	5
13334	W12c19	Knife	(4/3/1/8)	9.6	0.59	2	2	28	4		W24e62	Header	DOM: THE	311		W2 tw68	Header	DANGTE	11.5	8
W14x550	W12x19	Kristy	CASSIN	4.6	650	r				e	W24x62	Header	01/340401	11.5	ě	W2 to//8	Header	LhdnG16	11.5	2
	WINTE	Code	Uwacitii	58	650						WZ4MSZ	Header	13/9/216	11.5		WZ INSB	Header	Chchig16	115	,
W14x730	W24c76	Truss Che		T.	33	W10:26	Colt	\$4c2x30	8.5	0.59	W36:210	Header	2424578	51	2.5	W/36x210	Header	FF 745/8	25	3
	W24076	4	F14.25x172	E:	+1	÷	*	+3	*1	ť	W36/210	Truss Cris	*0	ři	*	W36x210	Trusts Crox	80	+3	*
W14x710 + 7Wdb FFs	FFS W24676	Trust One		3						÷	W39/300	Truss Ons				W39:300	Truss On			÷
12.543	W24c76	£	84,25492	ĸ	100	38	æ	398	Œ	æ	Whe31	Truss Cres	38	*		W8e31	Trass Cros	38	œ	ð
W14x720+2Web FTs	PTS W24c76	frust One	÷	10		9	65	32	£,		W39/300	Truss Ora	60	12		W36c300	Truss Ora	60	9	ŧ
	The second second															Total State of				

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity

Where the beam size is indicated no member is farming into the count of an area of a manual steel shop drawing for that connection

The information specified for each connection was taken from the datest corresponding Frankel Steel Limited steel shop drawing for that connection

The information specified for each connection was taken from the dates from the consequence of the connection of the con

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(1) Chloron (1) Electric (1) (1) Chloron (1) (1) Chloron (1) Chlor	<u>\$</u>			Bree   Bree   Without	10 for the control of	60 to 100	14.5 14.5 17.5 17.5	fector) 0.59 1.21 1.21	Bean W23566	Type Header Header	See 1340-516 1340-516	17.5 17.5 14.5	Feeta <sup>7</sup>	Esen. W20x116 W24x146	Fyse Header Header	See Lbosg16	Length [m]	Ε.
				1971601 197160 197160 197160 197160 197160 197160 197160	6.04 (5.04 (	(4ch)))	14.5 17.5 17.5	820 101 121 121	W23464 W24408	Header	Dang16	14.5	550	W24x146	Header	Likhigité	145	
2		50 450 3 450 4 503 4 4 5 6 6 7 503 504 60		70,7160 70,72465 70,72465 70,72465 70,72465 70,72465 70,72465 70,72460 70,72460 70,72460 70,72460 70,72460 70,72460 70,72460 70,72460 70,72460	6.64 6.64 6.64 6.64 6.64 6.64 6.64 6.64	(46.53) (46.53) (46.53) (46.53) (46.53) (46.53) (46.53)	17.5	201 121 121	W24509	Header	03/9/00/01	395		W24x146	Header	200000000000000000000000000000000000000		
				197465 197465 197465 197465 197465 197465 197460 197460 197460 197460	604 604 604 604 604 604 604 604 604 604	(60.00) (60.00) (60.00) (60.00) (60.00) (60.00)	30	121	20,000	Tables .				The second second		L3x2m5/16	14.5	
* * * * * * * * * * * * * * * * * * * *				92465 92465 92465 92465 92465 92465 92465 92465 92460 92460 92460 92460	6.64 6.05 6.05	(4ch)() (4ch)() (4ch)() (4ch)() (4ch)() (4ch)()	541	121	M8-61	Header	LACACTIE	a	0.0	W284230	Header	14e3clt2	8	
	* * 0 * 0 * 1 * 1 * 1 * 1 * 1 * 1 * 1 *	7		707465 707465 707465 707465 707465 707460 707460 707460 707460 707460	COAP COAP COAP COAP COAP COAP COAP COAP	(4ch)() (4ch)() (4ch)() (4ch)() (4ch)()	200		40' Side	Header	UND45/16	205		W27x84	Header	L3x3x5/16	145	
				924655 924655 924655 924655 924655 924655 924650 924600 921460 921460	664 664 664 664 664 664 664 664 664 664	(100 m) (100 m) (100 m) (100 m) (100 m)	17.5	171	40° Grides	Header	C34294740	205	×	W27x64	Header	13×3×516	345	$\neg$
3 4 7 3 4 7 4 7 4 7 4 7 4 7 4 7 4 7 4 7	0 100 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	3 1 227 1 3 3 1 3 1 3 1 2 1 2 1 2 1 3 1 3 1		92465 92465 92465 92465 92465 92460 92460 92460 92460 92460	Gobb Gobb Gobb Gobb Gobb Gobb Gobb Gobb	((CA))) ((CA))) ((CA)))	17.5	121	40' Goder	Header	tio information Available			W27x346	Healtr	130676	17.5	
				924655 924655 924655 92465 92460 92460 92460 92460 92460	Goth Goth Goth Goth Goth Goth Goth	(40)00 (40)00 (40)00	17.5	121	40° Grider	Header	UNDAP16	205		W27s84	Header	LNANGTE	5#	
> > + > + + + + + + + + + + + + + + + +		277 1 4 2 1 4 5 1 4 5 1 5 1 6 5 1 6 5 1		92465 92465 92465 92160 92160 92160 92160 92160	0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	Lechilli Lechilli	2.2	123	40° Gudes	Header	No information Available	+		W22x146	Header	13/3/6/16	17.5	
× × × × × × × × × × × × × × × ×	# * * * * * * * * * * * * * * * * * * *	0 1 2 2 2 2 2 3 3 3 3 3 3 3 3 3 3 3 3 3 3		92455 92455 648371 92150 92150 92150 92150	0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	140 20 310	11.5	1.21	40' Goder	Header	315/010	20.5	ĸ	WZZNBA	Heador	21/29/218	345	
× × × × × × × × × × × × × × × × × × ×		7 - 7 1 - 7 1 - 7 1 1 1 1 1 1 1 1 1 1 1		4 M 6377 4 M 6377 4 M 1460 4 M 14	5 5 5 5 5 5	And the same	3.0	123	40' Grides	Header	13x3x5/16	205	35	W22x146	Header	Disp.\$16	17.5	
* * * * * * * * * * * * * * * *		* * * * * * * * * * * * *		20 M 6377 12 M 630 12 M 630 12 M 630 12 M 630 12 M 630 13 M 630 14 M 630 15 M	Code Code Code Code Code	16000	3.0	121	40° Grafer	Header	31,040,10	20.5	10	W27584	Header	11/0/0/11	14.5	
X X X X X X X X X X X X X X	3 * 3 * 5 * 5 * 5 * 5 * 6 * 5	2 1 2 1 1 1 1 1 1 1 1 1 1 1		021450 021450 021450 021450 021450	Cate Cate Cate Cate Cate	Levell	20.5	138	40' Godes	Header	9159010	202		W36e170	Heador	05000	38.5	
* * * * * * * * * * * * * *	******	1 - 1 1 1 1 1 1 1 1 1 1 1		02150 02150 02150 02150 02150	10 0 4 4 0 0 4 4 0 0 4 4 0 0 4 4 0 0 4 4 0 0 0 4 0	(4c3c)(a	345	1.00	40° Grifer	Header	United/16	205		W24s76	Haader	13x345/16	11.5	
× × × × × × × × × ×				921450 921450 921450 921450	1 1 1 1	Lectriffi	14.5	1.00	40' Goden	Header	Undergre	20.5		W20x146	Header	1342438	14.5	
* * * * * * * * * * *	* * * * * * * * * *	1 5 5 5 5 5 5		921450 921450 921450	Costs Costs	0(000)	14.5	1.00	40° Siefer	Header	Dia4916	20.5		Workers	Header	DOM: 16	115	
	* 08 * 08 * 08 *	F1 1 1 1 1 1 1		921450 921450	Code	(4chill)	345	100	40' Soler	Header	13/0/6/16	205	180	WZW146	Header	12/2/18	14.5	
* * * * * * * * * *	3. 1. 2. 2. 1. 1. 1. 2.			V2145.0 V2145.0	Crite	(technill)	34.5	1,00	40' Goder	Header	134346716	200.5		W24e76	Header	11/0/6/16	115	
	C 2 X C 2 X		+	42160		(Incha)	316	100	40° Grider	Header	Us/04/916	20.5	*	W22c146	Header	UningB	17.5	
	2 * * 0 *		+		fish.	(6chill)	376	100	40" Gerier	Header	Unastric	205		W2764	Header	13/0/6/16	14.5	
	* * * *	- ,	+	W2540	Confe	Lechtilli	311	1.00	40' Sinter	Header	UNDAS/16	205	,	W24x146	Header	13-2-28	14.5	
4.70 E 6.00	**************************************			W23450	Cook	Lechsills	14.5		40' Grefes	Header	LNONSTR6	302	-	W24s76	Header	13/0/576	11.5	
T E E S			-	W2150	Colt	(6008	34.5	1	40' Griber	Header	DOST	98		WZWANE	Header	Ukor38	345	$\neg$
EEE		,		-	Colfe	(echill)	348	100	40' Grider	Header	1349474E	502	2	W24e76	Header	31/54047	544	
E.O.		,	+	-	Krafe	14c3c30	145	100	40' Girder	Header	13/3/6/16	20.5	8	W24x140	Header	00/c/c1	345	
A STATE OF THE PARTY OF THE PAR				0/017/4	100	0.00,00.00	2	001	*80.7M	Made	Local 16	15.8		0.7457.0	Heater .	9,000,000		
			+	WISOS	SNR.	Lectures 1	11.5	0.79	WXXXXX	Header	13x3x316	17.5		962228	Header	Livings	17.5	
+:	*:	100		W14622	Cath	Section	69	0.09	W471594	Header	DX20276	14.0	*:	W24x76	Header	CDGDG 16	14.5	
Whaness			+	W14622	Sus.	14003	972	0.00	W27/84	Header	1309236	14.5		WZPs946	Header	Lboras Liveras	244	
	,	,	+	W14622	SNN.	Lectrill8	88	650	W27884	Header	CICASTE	45	٠	W24876	Header	Christof 16	11.5	_
Without	6.53	ess		Meach)	100	(4,549)	9.0	0.00	MO Public	Meader .	11/2/2016	97		MOANTE	Design I	13074014	100	
			t	Weard	Total Contract	Lay by 100	36	0.58	WOOdd	Header	Dynamic	375		Withten	Header	136203	375	_
White/005			t	Williams	Code	Lechalli	99	0.59	Wilhell	Header	D0676	17.5		W24s76	Header	DOS-16	118	_
		.,			Seated				W36x170	Stated			2	W26x140	Header	13/3/38	345	_
W148730				H	Knik	Lectoritis	8.6	0.59	W24476	Header	UNDAR76	11.5		W24s76	Header	Lhdsg16	11.5	
				W14426	Colt	(echilli	9.6	0.53	W24s76	Header	UND-\$716	115		W24x146	Header	Descrip	14.5	
A SANCIA	,	,		W14426	Cally	(ecc)	9.0	0.69	W24s76	Header	0.004710	115	2	W24s76	Header	LINDAGINE	11.5	
WH4x730+2W45FFs	*			W14s26	Colfe	(4chill)	9.0	620	W24x76	Header	13434516	11.5		W28x166	Header	1343439	14.5	
12.54	œ.		0	W14s26	Knife	(4/3/3))	8.5	620	W24s76	Header	DANGH	115		W24x76	Header	DOMEN	11.5	
W16x230+2M46 PYs			× 1	W14s26	Krafe	(4424)0	4.5	0.59	W24x76	Header	13×3×6716	11.5		W24x146	Header	13/3/3/8	348	
			+	+	Code	(technill)	9.5	0.59	+	Header	13434516	11.5		W24n76	Header	Chchiggs	11.5	
W14x730 = 2Web PTs W19c26 Knife	- W			+	Truss One		+ 1	35		Dauble File	F1453472	345	35	W36x300	Deable Fig.	P145x1/2	49.5	
+		,		+	£	M 4 20x 1/2	15.25		W30x210	Truss Chin	*:			W386210	Inglis Cox	86	÷	
W14x710 + 2Wdb FF3				+	Trust Chs.	,				Truss Cas				W39c300	Truss Ons			
175015	,	,		W24e76	£	R4.55x17	g :	20	W36c280	Trusts Cres	×	÷	*	W398260	Trass One	×.	98	
In the cent Expertence Conf.				W24s76	£	FI 13r1	7.5			Instit On	0				Trans Con	9	6	
H H	it.	7		+	1		i e		+	7	7	+		*	+	it.	+	
Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	ction corresponds to	ratio of d	epth to th	e 14.5" de	ds to ratio of depth to the 14.5" deep connection analyzed	n analyzed by Ingra	ffea for ax	ial capaci	y and stiff	ness								

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W Weld I W Well I W W		[with [w]	Factor	Beam	Type	5.	Limpth Drill	factor?	Beam	1/24	5.	[within]	Factor?	Bram	fyje	Sor	[withful	Factor
W27665 W27655 W27655 W27655 W27655 W27665 W27665 W27665 W27665 W27665 W27665 W27665 W27660 W27660		100 8.5	0.58	W14s22	Code	Lechalli	89	0.59	W24nt3	Header	13x3x576	115	53	W24nd3	Header	Lbchs716	115	55
W2465			100	W18-05	Krish	14c3x330	115	0.78	W3049	Header	13/34/216	23.5	æ	W24x146	Healtr	13/04/216	345	*
000450W  Waters Statement of the control of the con		175	121	W18e3S	tot.	(4e/s/3)	115	0.78	W2049	Header	13434576	395		W24x84	Header	Docts	17.5	*
Wands			17	W16426	Colt	Lechilli	9.2	050	W27/84	Header	L3x3x5/16	14.5		WXXXXX	Header	L3x3x5/16	20.5	
W2465 W2465 W2465 W2465 W2465 W2465 W2465 W2460 W2160		17.5	17	W16426	Cnife	Lech 300	8.5	0.53	W2264	Header	C3/29/210	34.5	æ	W24srk9	Header	915000	115	35
W24455 W24455 W24455 W24455 W24455 W24455 W24455 W24450 W21450 W21450			171	W16/26	Cott	163038	95	450	W2344	Header	DOSE	145		WZ4x162	Header	thosp16	345	*
W24455 W24455 W24455 W24455 G4444 M8371 W21450 W21450 W21450			121	:W16/26	Kel9.	14×3×3/8	8.5	0.59	W27-84	Header	DAME TE	14.5		W24s68	Header	LP-D-Stre	511	3
W2465 W2465 W2465 Gude MB2F1 W2160 W2160			121	:W16/26	Kriste	(4e/3/3))	18.5	0.59	W2344	Header	0.0900	17.0	*	W24x162	Header	13/0/616	345	15
W2465 W2465 Gide M8371 W2160 W2160		17.5	121	W16/25	Code	Lecht28	8.5	0.58	W27484	Header	21,540,01	14.5	2	W24s68	Healor	139948	115	1
W2465 Gude MB371 W2160 W2160			128	W16426	Colfe	14c3c30	8.5	0.59	W27r84	Header	Uk05/16	3.5	35	Wzwiez	Header	UNDHQ116	316	3.
Gude MS311 W21450 W21450	# DESCRIP		128	W16/26	Code	16000	978	050	W27x94	Header	31,040,00	145	35	WIBGE	Header	DOMEST	11.5	
02+CW 03+CW 03+CW	A LEAGUE	302 97	141	W16/26	Code	160039	98	0.59	W27694	Header	0.694540	14.5		W/36x150	Heador	DOSTE	392	*
W21450 W21450	RECKLIS #		100	W1626	Code	(6c3x1)8	98	65:0	W23x94	Header	UA98416	14.5		W24s76	Header	13x3x5/16	115	3.
9021450	fe L4c2c3/S	345	100	W16/26	Sub-	16:3:38	185	0.59	W23x84	Header	13/34/216	145	-0	W24c162	Header	1342438	14.5	
	le Leadill	10 145	100	W16/26	Kin fe	1600038	9.5	450	W27s54	Header	13/2/61	14.5		Wowne	Header	Lhoighte	115	3
W25650 Crafe	M, Lkobolin		100	W16/26	Kriste	0(1429)	8.5	650	W23x84	Heater	31/9/017	18.5	*	WZW162	Heady	00000	145	*
W2160 Cuite	# LACROIN	216	1.00	W16/26	- Crafe	16/3/33	8.5	650	W23x84	Header	13/24/216	14.5		W24e76	Healer	LHOMETR.	115	î
WZ160 Kish	techning the party of the party		100	W16/31	Soite.	\$60x38	50	0.59	W27s84	Header	13/3/6/13	14.5		WOOM32	Header	Lavangins	302	ar.
#2150 Golfe	# DENDUM	314 145	100	Witertsti	fish	(dract))	29.5	2.03	W27/64	Hade	13/39/216	14.5	- 65	W26c160	Header	DOMETE	382	5
W21650 Knih	8. teadls	.514	100	W16426	Knft	techilli	9.2	0.59	W27x84	Header	DAD46710	14.5	,	Wzwiez	Header	UNNUM	145	
W23450	H L4x3x3/B		100	W16/26	Civite	Lechalls	82	0.58	W27594	Header	13/25/16	14.5		W21s57	Header	1304576	11.5	*
W2145710 Code	1443433		100	W16k26	Code	160038	8.5	450	W03/84	Header	DOST	14.5		Wzeciez	Header	Ukoras	345	53
W21450 Knife	\$ (4-3-1)%	145	100	W16/26	KNS.	14c2c38	8.5	0.59	W27494	Header	13434576	14.5	2.5	W21557	Header	13x3x5/16	411.5	
	54 L4x3x3/III		100	W10/26	Krite	L6c3x300	45	0.59	W23/84	Header	01/04/010	14.5	(6)	W24x162	Header	13-2-19	345	.2.
W21460			1.00	W1626	to the	Lectural	9.5	0.09	W23584	Header	11504216	14.5		W21x57	Healer	LPoM/16	HS	
W14x426 W14x22 Code	# L4c2c39		629	Wt2x19	Crafe	Likerige	26.5	183	W36x360 ··	Header	13x3x6716	28.5	æ	10/24×16/2	Header	UNIVERSE SE	34.5	ं
W14422	4 (4c3c3/l)		0.59	W12519	Knife	14c3c30	912	0.58	W27584	Header	03/25/07/0	14.5	*	W21552	Header	Chebane	11.5	£3
			650	W12s19	Co.fr	14c3c33	9.5	0.69	W27484	Header	13/3/63/6	14.5		W24x162	Header	18-9-38	346	
+			620	Wt2st9	599	Lechtijg	8.5	650	W24x76	Header	Ukbey16	115	*	W21=67	Header	Lbc34516	11.5	*
+			650	W12e19	Kesh.	14-2-12	8.5	650	W24x76	Header	13:04/36	11.5		Wzkatez	Header	13-3-38	17	5
W14/22			650	W12519	Code	(40.00)	98	650	W24476	Header	DOM/16	11.5	,	W21957	Header	UNDAGTI6	118	1
+			600	Wikits	Co.	Lacksije	88	0.00	W18et7	Header	1309216	311	-	W24x162	Header	LN2x39	34.5	•
+	3		0.00	W12319	200	Cochago	0.0	650	W.200.508	Header	0.555.00	17.5		W2 1857	Header	Chronic 16	411	1
Wiscon march perce				W12019	500	1602030	912	0.00	Octobe 200101	Heater	0.000000	225		10746.302	Healtr	130338	4.5	2
Wischel	-	98	0.69	Wibris	Cole	(actual)	98	0.69	WOTACT	Header	Livingsin	118		Withhit	Heater H	Ubacili	371	
H			0.00	W12x19	570	Lachilli	98	0.69	W21467	Haufer	(3/0)/(3/0)	311	2	WS takin	Header	Chobdys	311	2
WTec26			650	W12x19	Co.tr	(400.00	8.5	0.08	W21x67	Header	1369676	11.5		WZWANG	Header	1343438	14.5	
H			620	W12x19	Knife	(6/3/3))	8.5	0.59	W21s67	Header	DOMEN	115		W21s67	Header	Dongra	11.5	
WHACE Kuity	F. LASSIN		650	W12x19	Knh	16(3)(0)	4.5	0.59	W2167	Header	DAM/216	311	8	W24x162	Header	00-00 C	345	*
WHACE Knits	N LACASIS	58 50	650	W12x19	Code	(Acadilli	9.8	0.59	W21457	Header	UNDAG16	11.5		WZ1457	Heater	DOMETE	1115	
#				W24c76	Truss Cox		+	35	W24x36	Header	13x3x5/16	115	3.5	W24x16Z	Header	13x3x5/16	14.5	3.5
	Predictory Predictory	15.25		W24s76	£	F14.25x192	21.25		W21s62	Header	D10M216	11.5	*	W24n62	Healer	13/04/16	12.5	*
W24c76 Trust Ces		.9		W24s76	Trust Cres	.9	.6	ě	W30k350	Header	13/9/216	28.5		WHENTED	Header	Dobin	28.5	
W24c76	843543	1525		W24c76	£	Pl 4 25×172	z		W32x130	Header	UN245/16	23.5	::	Wile31	Header	030476	5.5	d.
W34c70 W24c76 Institute			ç	W24s76	Ings On	2.0	£		MS-31	Header	13x3x1/4	23.5		Sedm MS31	Header	13/24/216	22.5	87
W24476 Fin	A 25x12	42 21		W24s76	¥.	Ft 2 25x334	12		W21x50	Header	DXD6716	11.5	i÷.	W3069	Healer	UND9216	17.5	
Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	knife connection c	corresponds to ration	of depth:	to the 14.5'	deep conner	Is to ratio of depth to the 14.5" deep connection analyzed by In	graffea for	axial capa	city and sti	ffness								

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C5.14

JA-4140

COLUMN 72	
YPE CATALOGUE -	
N CONNECTION T	
INTERIOR COLUM	
C5.15	

			North.					- South					East					West		
Column	Seam	Tipe	#E	length [in]	Factor?	Beam	1,0pt	5.	Limpth Drill	Factor?	Ben	1/04	5	[u] qubus .	Factor?	Eram	fype	155	[w] quban	Factor
W14d0	WHACES	Code	LACACITIS	8.5	0.58		-				W21x73	Heider	31315118	115		W24x62	Header	3×3×5/16	115	55
100.000	WIBOS	Krife	(4x3x3/8)	11.5	420	0.	0.			æ	40" Sinfer	Header	13534576	23.5		W24x62	Header	31/54040	14.5	*
609400	WYBASS	Chile	SEASON SE	111.5	0.78			*	+:		Grafe MOS1	Header	44347716	a		W26k150	Header	4c3c30	ø	
And Articles	W1626	Cole	(4che))))	982	650	,	9		4		40' Sider	Header	UND45710	20.5		W27x84	Header	DAD-6/16	5#	
W194x120	W16/26	Knife	L4x3x3/B	8.5	0.50	3%	.8	36	:6	æ	40° Grides	Header	03/3/6/10	205	æ	W24x68	Header	0.00/g16	115	*
1000,000	92958	Code	14436333	8.5	450	45		4)	£		40° Grider	Header	Meletemator Available	-		W24e58	Header	No information Available		*
41.148.137	W16/26	Knis	E4x3x3/18	988	650	2	2		3	æ	40° Suder	Header	Living'se	312		W24s68	Header	Na Information Available		2
	W16/26	Krife	Lechelilli	9.6	0.50	Ж		*	÷	8	40° Girder	Header	Holinformaton Available	Ŧ	*	W24s/A	Header	Nutribemation Available	20	*
W14x123	W16/26	Code	Ldx 3x 3/3	8.5	0.58	,		100	÷		40" Goder	Header	345/017	20.5		W24e68	Headov	Walnishmaton Available	-	1
100.000	W16/26	KssNe	(4x3x3/8)	382	650	es.	38		e e	38	40° Grides	Header	UND-\$716	23.5		W24x68	Header	0.09676	115	3.
10 14x 133	W16/26	Colfe	(4e3e3))	RS	0.50	8					40° Grafer	Header	DOMESTIC	302		W24x68	Header	31/5/0471	11.5	*
Marchane.	W18/26	Code	Deball	98	650	,					40' Godes	Header	0.000	205		W24x68	Header	0.00416	115	•
0.146233	W18/26	Code	Lackelffl	38	650	×	×		3	2	40° Grifer	Header	United/16	23.5		W24e88	Header	3129-0101	11.5	.1.
the same	W16/26	Colt	L4x2x3/S	8.5	850	5	5				40° Soller	Header	Uspegre	215		W2468	Header	DOST	11.5	*
0.146,033	W1826	Knife	MDGMI	910	650	A	1.5				40° Sieder	Header	31/3000	205		Works	Header	31/50017	115	3
1000	W16/26	Chife	Like Sectifity	9.0	050	x	8		÷		40' Goles	Heater	13/0/01	20.5	*	W24e68	Header	Dogse	115	*
W166311	W1626	Code	LACTOR	912	650	,	,		1.		40° Goder	Header	0304216	23.5		W24x68	Header	Liorette.	115	1
100340363	WISG1	Code	Debolis	9 89	0.58				+		40" Girden	Header	13/3/6/16	23.5		W24s76	Header	Dodyte	115	ar.
Par sales	W36x150	Knite	(fowed)	29.5	2.03						40' Grife	Header	13/3/6716	205		W33k130	Header	Changité	312	*
OKC-95(8)	W16/26	Knife	teach	88	650	,		9	9		40' Side:	Header	13×3×670	225	,	W24x62	Header	D/D/5/16	115	1
	WTS/26	Crite	L4x3x3/B	38	650	*			9(		40' Sinfes	Header	Dong16	23.5		W24e62	Header	LNOSTIS	11.5	÷
10114-718	92900	Code	Lécholis	8.5	650		-		*		40° Grider	Header	DOSTE	205		W24n62	Header	Dogts	115	*:
	W1626	Knife	SACHAR S	9.6	650	2	2	2		2	40' Grider	Header	13434516	202		W24e62	Header	31/94047	115	3
3674-64790	W16/26	Knife	L4x3x3/III	9.6	650	8	9	20	*	8	40° Girder	Header	13/3/6/10	23.5	(4)	W24e62	Heisder	13/04/216	11.5	2.
	92,014	Colt	L4x2x3/5	50	650			E	-	,	W28x71	Header	13/24/216	11.5		W24e62	Header	DOM:016	118	1
W14x455	W33k130	Coste	L4x4x3/B	23.5	1.62			(if	74		W33x130	Header	13/29/216	2115	æ	W24x62	Header	Displays.	11.5	ं
	W12x19	C18	[4c3c3/8]	8.5	0.59	15	10	*1	10	*	W24s62	Header	01/01/01/0	211	*	W24x62	Header	915600	11.5	*
W146500	W12x19	£uş.	(4chc)/8	8.6	650			t	,		W24x62	Header	13/59/31	11.5		W24x62	Header	1309576	115	
	WINE	200	Lexivities	3 8	650		,	*			W24x6.2	Header	DISSENT.	115	٠	W24m62	Header	Christine	11.5	*
W14e550	WINE	Code	LACTOR	20	650				e o		W24e62	Header	13:04216	11.5		W24s62	Header	Change	1	5
	W12019	tun.	Decision	4 0	0.50			+ 3		,	WZ4652	Header	Donath	135		WZ480Z	Header	Changes	113	1
W14x550	0017010	470	140,0010	0 0	0.00						100,440	Hander	Table Day			Witness	Header	1 Policing	2 4	
	901348	4.0	140,000		0.68						Withelet	- Indian	13-09-6146	***		On the same	- Transport	130,636	344	
W14605	W12419	Colfe	LACACIA	345	0.00						Wheels	Header	(34345)6	1116		M0.142	Header	Chengue	N.	1
	Wilkers	Colk	L4s2s3ff	8.5	650						W24x62	Header	UNB4716	11.5		W34x62	Header	0.00676	115	ľ
W14e605	W12ct9	Code	LACAGIL	8.5	650	×		200	9	æ	W24x62	Header	03/39/30	11.5	20	W24x62	Header	0.004010	115	25
1000-1000	Wrzetb	Code	LACKSASS	9.0	0.59				*		W24n62	Header	134246716	11.5		W24n62	Header	Changing	11.5	8
W148085	Wilkers	Knife	(4chill)	8.5	450	2.	2			22	W24e62	Header	DAMAN	115		W24n62	Header	Dishiption .	511	3
40.00	W12x19	Krafe	Uschilli	4.6	620	,	,		÷		W24x62	Header	13434670	211		W24x62	Header	0.09000	11.5	2.
CONTRACTOR OF THE PERSONS	W12c19	Colte	Uschilli	50	650			(4)			9024462	Header	13/09/216	11.5		W24n62	Header	Changes	115	1
00.000.00	W24e76	Truss One			35	W10:26	Code	14424310	8.5	0.59	W36k210	Header	Physics	22	35	W36k210	Header	977-578	21	33
	W24-0'6	4	FL4.25x172	11	+1	1		+3	70		W30x210	Truss Cris	*	70	*1	W36x210	Trusts One	80	+3	100
W14x720 + 2Web FTs	FS W24.06	Truss One								÷	W36/300	Truss Cas				W39x300	Truss On			
125415	W24c76	£	N 4,25x V2	£	136	36	æ	39	9	æ	Whr31	Truss Cres	36	:		W8e31	Trass Cro	×	96	ð
W14x730+2Web FTs	rs W24c76	Tress One	r)	10	6	9	9	320	£		W39c3co	Trust Que	65	#		W36c300	Truss Cree		9	*
																Total State of				

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity

2. Where no beams size is indicated no member is faming into the county and a The information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

3. The information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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MN 73
LOGUE - COLU
N TYPE CATA
IN CONNECTIO
INTERIOR COLUN
C5.16

Column	9	Starn Type		5	Length Sel	facto,"	Stam	hpt	504	Length Srid	Factor*	Bram	Tape	101	Longth End	Factor	Seam	Type	Nee	Length [in]	Factor*
sed W14c99	H	-		(4c)c)t)	47.5	121	WIEGI	Code	14c 3c 38	88	650	W27484	Header	UNDAGTS	44.5	-	W23x84	Header	(34)46/18	34.5	
Sapr 47	1	i e					W21e50	Knite	14<2<3/8	34.5	100	W24m62	Header	134245716	34.5	100	W24s68	Header	25/34040	343	<b>S</b>
300:46		*					W2465	Knife	14e3c38	17.5	121	W36-210	Header	Lechold	305	4.	Geder MG61	Header	144317/15	22	÷
3ser 45 W16=146	75	1			4		W21650	Crite	(44343)	14.5	100	W24676	Header	13:05/16	17.5	4	40° Goder	Header	Unde\$116	305	
Auge 44		4			ī		W25450	Code	1443030	34.5	100	9/24/26	Header	DOMEN	37.5	56	40° Gisper	Header	13x3x6/10	30.5	ä
Sur 43 W34:75	2	*			,.				10				0		-		,	+			+
		2		9	-		W25450	Links	LAchold	34.5	100	W24e76	Header	Dodys	47.5	100	40° Gister	Header	13-05/16	30.5	4
Rape 41 W14e211	35	£		e						v						120		,			÷
					,	0	W21x50	Knite	(Achi)§	14.5	100	W24e76	Header	0.04916	12.5	×	40° Gisder	Header	13/3/6/18	20.5	
3ser 39 W14e 202	0	er er			,	0	W21650	Culte	(4c)c3)3	14.5	1.00	W24476	Header	13/04/16	12.5	i.	40° Gister	Header	131016/16	302	¥
		1					W21660	Enle	(4434)))	346	100	365450	Header	0.04914	17.5		40' Gister	Header	0.000	20.5	÷
See 37 Write-342	9	4				4	W21x50	Crite	D40038	14.5	100	W24476	Header	130/216	17.5	-	40' Girder	Header	11/5/04(1)	302	. 4
See 16		*			4	,	W21450	Coale	144242/8	34.5	100	9/544/26	Header	1306516	12.5	ų.	40' Girder	Header	13x3x5/16	302	+
Reer 25 Wyay Sea					÷		W21450	Cook	1443438	14.5	100	W24476	Header	DOSTI	17.5	2	40' Girder	Healer	13/94/38	20.5	-
Sper 14		*					W21650	Krife	LAchegia	14.5	100	9/24/26	Header	13x3x516	17.5	502	40' Gister	Header	13/3/5/16	305	
Bar 33					÷		W23x50	Knife	14e3e39	345	100	9/54/58	Header	13/29/216	17.5		40' Girfter	Header	13/045/16	502	+
Base 32	8	F. (4)			ř	,	W23560	Knife	1442638	14.5	100	92458	Header	Lbody6	42.5	ï	40° Girder	Header	13x3x6/18	305	1
3se 31					-	04	W2360	Kuite	(44.34.38	34.5	100	97/8/28/	Header	134345746	12.5	a.	. 40° Gister	Header	34/540401	305	
Base 30					,		W25650	Knile	L4e2bc3/9	14.5	100	32458	Header	DOME	17.5	ě.	AU' Gister	Header	(3/5/0/2)	305	+
Roor 29 Misserro	5	3			3		W2 5450	Crite	(44343)	14.5	100	W24c76	Header	1305416	17.5	ű	40' Goder	Header	54.54PE	302	4
Rase 28					-	×	W23/60	Corfe	1443438	14.5	100	W24476	Heater	13:04/216	17.5		40° Girder	Header	13/0/28	300	÷
See 17 Misselle					,		W25-50	Krife	L4<3×3/8	14.5	100	9,040,6	Header	DONNE	17.5		40' Goder	Header	DOSS	2015	+
Base 26				1.0	1	14	W25450	Links	LACIONS	14.5	100	W24s76	Header	Dogte	47.5	33	40° Gister	Header	13/24/16	30.5	14
Reer 25 WY Gurdon					٠	٠	W21650	Knite	LAcheMil	14.5	100	W24+7.6	Healer	13/24/214	17.5	-	40' Gisünn	Header	13/0/618	20.5	
Rose 24					,		W24c76	Knite	(Ac)c()()	17.5	121	40° Soder	Header	DOMPE	225		W27x84	Header	LPsh4216	5.43	
See 23 Wester	,	98				38	Goler MB371	Colle	14ch38	20.5	141	Alf Girder	Header	13/04/14	20.5	9	W27/64	Header	130696	17.5	24
						٠	M21450	Loke	(4c)c)()	14.5	100	40' Sirder	Heder	DOME	22.5	i.	W27/54	Header	0.040.0	345	
Reer 21 W144/230 + 2W46/FTs	Web Fix				ŧ		W21x50	Knife	(4c)c)(8	14.5	100	40° Goder	Seated				W27rd4	State.	Lincolas	345	100
3ser 20 12 Ser					9		W2360	Knite	LAchcill	346	100	40° Girden	Seated		i i		W2 hds.	69.96	144243/8	14.5	1.00
WHAZ	Web Fig.	+		÷			W2550	Keele	[443-33]	14.5	100	40° Soder	Sexted		¥.	40	WZ3nlis	full	[4x2x3/III	385	100
Raar 18 12-5-2	2	9		Œ	.,		W25-650	Knit.	(4c3c3)0	34.5	100	40° Godes	Seated	54	ő	14	W23v84	Cork	(4x3x3/8)	24.5	1.00
Soc 17 W14+230 + 2866 FFs	Web Fits	*		×		×	W21.50	Knite	(4c3c3)8	345	100	4cf Goder	Seated	(4)	æ		W27n64	Crite	144/3/3/8	14.5	100
					,		W2465	Knite	(4c2c38	11.5	121	40° Soder	Seebed	27	2		W3059	fork	144,3438	17.5	121
Pair 15 W14x710 + 2Web FF;	Web II's			×			Goder MG152	Knite	1443138	302	1.63	40° Godes	Seated	Œ	4	(S)	W24s76	Seated			*
		•					W16e60	Corbe	LAc2c38	9.5	0.53	40° Birder	Seated				W24s74	Enth	14x3x3/8	345	100
Reer 13 W14x/120 + 2W46/H1	Well's	3		×			WIGHE	Crist	LACTOR	9.5	650	40' Godes	Seated		i		W24s76	Soft	[46263]	14.5	100
		•				6	W10#0	Keafe	(4cht)0	50	0.59	40° Goder	Stated	-		S.	W24s76	fatt	[4x2x3/8	24.5	100
1966 11 W342330 + 2864 FF)	Wet Fris						100,040	200	LACTOR 200	5 40	600	40 Grider	Jeans Control				W24K75	100	[44,26,38]	14.5	8 9
	100.00						Wideko	Code	(Achelli	98	0.08	ACT Globa	Sealed				WDANTE	Total	1407038	346	100
Tant 8 125.65	S						W16x60	Knite	(Arbchill	9.5	0.63	40° Soder	Seated				W24c76	Code	[44/2628]	388	100
	┖	W16c26 Forte		[4x3x3]	8.5	650	W24c76	Truss One				W36-210	Knife	(Averal) - 3/8 weld	46.5	388	W36:300	Dauble Fin	F145x1/2	38.5	1.69
Reerl 12548	9	Not Fn		R 10c34	10.		W24c76	Æ	PASATZ	18.25		WINGTH	No lefo	E	*	í.	W36-210	Trust Onc	**	i	+
Rest Whedgo zwelfts	Wells				9	3	W24s76	Fryss Clox	ě			TRUSS #1	Trust Dec		4		W39c360	Trust One	1.4		9
Rope 4 12 Sal	8				9		W24c76	Fin (Adopte)	F145x1235555000	15 (18)	•	W36x194	Knie	14-3-38	29.2	2.03	W36c260	front One			+
Reer 3 Ecotonia Col	Cal	+		E	+		Wzecze	E.	Pitht	345	12	100	÷		22	20	- 22	20	4	ï	+
		9		10				10		÷	14	9	9		i i	14	14	9	i.	i	4
Notes 1 Scale	factor lis	ted for each	knife conne	ction correspond	ds to ratio	of depth t	o the 14.5	deep conn	1 Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	graffea for	axial capac	ity and sti	ffness								
	ctor liste	d for other co	onnection ty	No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity	conservati	vely assum	ned to have	sufficient i	tensile capacity	,											
2 Where	* no hear	ne ciza ic indi	rated no me	2 Where no heams size is indicated no member is framing	a into the column	o de infor															
	2	2 2 2 5																			

74	
COLUMN	
CATALOGUE	
TYPE	
CONNECTION	
Ę	
COLUN	
INTERIOR	

				North				1	. South		-			East					West		
Column		-	Type	455	langth (in)	Factor?	Besen	Type	5	Limphibri	Factor?	Beam	Type	5.	[w] qubun]	Factor?	Bram	fype	204	[w] quban	Factor
Witedio		W14422 8	Cult	LACACITE	115	62.5	W16:31	Snt	C4x 3x 313	65	650	W22949	Heider	1,4×2×5/16	14.5	-	W24nfill	Healer	14×2×576	115	55
Are a story		W21450 K	CHR	(4x3x376)	14.5	100	W10-01	Coste	144,3130	3.5	0.58	W24s76	Header	144246710	14.5	æ	W30/69	Healtr	144345710	22.5	*
Contract of the Contract of th		W2465 K	Khite.	E4x3x3/B	5.4	121	W16:31	tot.	(fechal)	85.	850	W22494	Header	(4x2x5/16	596	-	W3069	Header	(4.04ST6	14.5	5
Market		W21650 K	Colfe	L4x3x3/B	14.5	1.00	W16631	Colt	Lechilli	9.5	0.59	W24x68	Header	L5x3x5/10 L4x3x5/16	14.5		W2754	Header	L4x2x5216	4.5	
Carin		W21-50 K	Colfe	14×3×3/8	345	100	W16-31	Code	1443-300	8.5	6.53	W24x68	Header	150479140476	345	æ	W27x54	Heater	(442)-5716	345	*
W14x122			+	17	÷	400			(2)	•	**				-	-			4.3	*:	*
0.00		W2160 K	Kn5	C4x3x3/8	14.5	100	Wiedt	Knife	14/3/38	8.5	6.68	W24s69	Header	LExhight best-14r3r5/16 best	514		W27x54	Header	LANDASTE	45	2
W14e176		÷		,	r	e	×	A		÷		è	1	*	Ť		+			X	150
100000		W21650 8	Colfe	U4x3x338	5#5	100	W1601	Cole	14/3/38	8.5	0.58	WIASS	Header	LSsheg/16 best-14s/h5/16 best	14.5	×	W23x84	Healer	1404916	34.5	2
10/14/10/		W2160 K	Kush.	L4x3x378	516	100	W16031	Kra Se	L4c3x30	8.6	0.59	W24x58	Header	L5x2x5/15 bent-[4x3x5/16 bent	14.5		W23x54	Healer	L4x3x5/16	516	8.
170611		W23-50 K	Cult.	(4chill)	388	100	W1601	Cufe	00000	8.6	050	W24s68	Header	15c2cf/16 best-14c2cf/16 best	345		W2354	Header	3159000	14.5	*
100000	H	W2160 6	Code	DADAIR	. 514	100	W1631	Cale	16/0/20	98	0.59	W24x58	Header	LEchsy16 best-14/3/4/16 best	345		W2754	Header	1404916	311	•
10.00	_	W2160 K	Cult	(ACAC)R	345	100	W16.01	Code	(4c3c)(8	8.5	650	W24x68	Header	LEAD-6/16 best-14x3x5/16 best	14.5		W2354	Healer	(4)04576	511	.1.
The state of the s	H	W2160 8	Cult	L4c2c378	14.5	100	W1601	Code	(4000)	115	0.59	W24e88	Header	L5x3x5716 boxt-14x3x5716 bext	345		W2754	Heater	(405)16	14.5	5
WHEN	_	W2160 K	Kuife	8/DO43	14.5	100	WIBG1	Code	8(E)(2)(E)	8.5	650	WZ4v68	Header	LSchrift6 best-L4c2+5/16 best	14.5		W27/54	Heater	Mostre	145	13
100		W25x50 K	Colle	LACACIAB	145	100	W16.01	Code	0(14791)	8.5	650	W24x68	Heater	L5x2x5/16 best-14x2x5/16 best	385	*	W27x54	Healer	UADISTI6	14.5	*
WINGE	_	W21450 R	Colfe	LACROTTS	345	1,00	W16/31	Crafe	(lectroll)	8.5	650	W24sti3	Header	USchell 16 best-14/2/15 best	14.5		W2754	Healer	(40/516	345	1
***************************************	H	WZ1650 K	Colt	Decksile	511	100	W16/31	Crite	(techol)	8.5	0.59	WZ4v68	Header	L5:315/16 best-14:315/16 best	5 %		W27554	Heador	1600676	345	*
1014678	_	W21450 X	Shik.	CACACITA	14.5	100	W16/31	fok	00 Ce1	8.5	950	WZanis	Heater	USO/G16 best-14/2/G16 best	576		W2754	Healer	(40676	14.5	*
1000		W2160 K	Knik	1443438	311	100	W16G1	fat	Lechalli	8.5	0.59	W24s68	Header	L5:0x5/16 best-14:0x5/16 best	14.5	,	W2754	Header	L4x3x5716	14.5	1
07400168	_	W23450 K	Chit	Lecholys	34.5	100	W1631	Crete	Lechal9	82	650	WZ4s68	Heater	LSignS/16 best-Michel/16 best	14.5		W27/54	Header	LAGISTIS	14.5	
307-75-00		W21450 K	4vo	14434378	345	100	W16:31	Code	(46.9038	85	950	W24×58	Header	LSx2nS/16 bent-l4x2nS/16 best	345		W27x54	Healer	LAGASTS.	345	*:
		W21-50 K	Knife	24574398	511	1.00	WEGST	Colle	14-7-38	. 59	0.59	W24=68	Header	LExchEpts best-14c0mEpt 6 best	5#4		W27x54	Heafer	14x345/16	5#	8
Withern	_	W2360 K	Krish.	54x3x378	545	1.00	W10/31	Kriste	145/21/20	.92	0.59	W24x69	Hesder	LSx2x5/16 bent-14x3x5/16 best	245		W27554	Header	14/2/6210	345	2.
		3534G6 K	Suh.	UkeZelifik	418	121	W10/31	Soft	(4c2c33)	9.5	-0.58	W0059	Header	Uschell 16 bent-LAr bright bent	22.2		W27484	Healer	LAXDAGTE	14.5	
W14v660		-	Cush	L4x4x378	20.5	1.41.	W36x150	Crafe	Likericke	29.5	2.00	W23x84	Header	LSc3c3/8 best-14c3c3/8 best	23.5	æ	W27x84	Header	LAcacala	29.5	÷
			410	(4c3c33)	34.5	100	W10.26	Cale	56-2-28	8.5	0.53	WQ3484	Header	15x3x5/16 best-14x3x5/16 best	345	*	W27584	Heater	14x2x6716	14.5	*0
WANESO	1	-	Kn9	LANDERS	3 1	901	W16:25	Cak	140000	9.5	0.69	W23484	Header	Eschill best-14ch(916 best	14.5		Wghile	Header	[4036]16	145	
	K .	+	200	L4x3x3t8	H2.	100	W18626	State of	Lectrific	9	650	W23984	Header	Example best-taxary16 best	14.5	٠	W24876	Header	(4x3x216	11.5	*
WHANGE	_	W23M50	Sun.	[44.3cm]])	2	100	W10c26	and a	Section 1	2 2	020	W22x84	Header	Language benefit and a find a	2		Waters	Header	CACAGOS CO.	1	1
2000000		H	Total Control	(arbeith	275	100	Webse	Total Park	(achille	96	0.68	WOMEN	Header	Hochelite hant (de hell) it has	370		W10-67	Header	Lacheria	311	1
W14x605	_	H	Culte	LACACIES	18.5	128	W1626	Code	CACALIER	8.5	650	W23v84	Header	L5x2x578 best-(4x2x576 best	14.5		WXXXXX	Healer	(405216	17.5	1
		Girder MG152 K	Cult	[4x4x378	MS	1.60	W16/26	Cole	(40.00)	8.8	0.59	W23x84	Header	LSGnG16 best-14c3nG10 best	345	2	Grider MOTS1	Heador	1606916	21.5	2
W148666		Witness	Culfe	LACALIB	8.6	0.58	W1626	Knie	8EX291	8.6	650	W234B4	Header	15x3x516 best-14x3x516 best	14.5		W21457	Header	(40676	11.5	*
	H	Without	Stalk	LACACAR	9.8	650	W16/26	Cale	(techall)	9.0	0.59	W27x84	Header	Updag16 best-14c3c216 best	14.5		W2167	Heafer	DADIG16	115	1
10 141300	_	W16s40 K	Cuff	teods	9.0	650	W16/26	Cally	16000	9.0	0.19	W23/64	Header	LSchill Sent-Mchil/16 bert	34.5	2	W21457	Heater	1400010	1115	2
0145-3300		WYGEGO	Crite	LACKSTR	3.8	450	W16e26	Enth.	(462638)	9.0	620	W23x84	Header	L5x2x6216 best-14x2x5716 best	14.5	٠	W21x57	Healer	LANDAGING.	11.5	*
		Withen	Knife	CACALIN	3.0	629	W16/26	Knife	(4c/s/3))	8.5	620	W27s84	Header	LSchStf6 best-14chStf6 best	514		W21457	Healer	14/21/216	11.5	13
00146730	-	WTG-60 K	Krish	CASS/III	4.6	650	W16/26	Krafe	(46.26.30)	4.5	0.00	W22x84	Header	LSchol/16 best-14cht/16 best	594		W2167	Heater	1406710	11.5	2
			Colfe	LANZATE	50	650	W16/26	Code	(4c2c3)	9.5	0.59	W23484	Header	USchill Sent (Arbrill Sent	12.5		WZ1457	Heater	LAXARITE	11.5	*
W14x730 + 2W4b PFs		W24c76 Tru	Truss Cher		1		W24x76	Trust Cox	36	+	35	W33x138	KNA	L4ch3/B	205	1941	W24s76	Colt	140009	11.5	0.78
125,65		W24-07-6 Dou	Double Fin	F14.25x172	18.25	+3	W24s76	E.	F14.25x172	2128	*	W21s62	b.	*0	30	*	W21x62	Self.	14000	11.5	679
W14x710 + 7Wth FTs		Н	Truss One				W24x76	Trust Ons	3		ė	GROSER 448	Truss Cas				Wabitto	fork	(4+4c29	285	2 03
12 5x8		-	Fin (+Angle)	R425412 8565478	15.25 (19)	13.	W24c76	Fin (+Angle)	R425412 (ISASANI)	21.25(52)	*	æ	ě	38	*		W22x130	For.	1444420	23.5	1.62
W14x730 + 2Web FTs		W24c76 In	Trust One	*	£	60	W24s76	Truss Om	32	+			**	60	12		MS-31	Colle	L4r4x278	22.5	162
-		100		Control of the Contro			Date A. W.	100	Contract Contract Contract	27.57.57.57							44000		1	***	0.00

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity

Where the beam size is indicated no member is farming into the count of an area of a manual steel shop drawing for that connection

The information specified for each connection was taken from the datest corresponding Frankel Steel Limited steel shop drawing for that connection

The information specified for each connection was taken from the dates from the consequence of the connection of the con

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

Victoria         Sipe	Type	5	Lemath Lini	1.00			å	Lanceth Lat	E. real				
WHORD I         CORA         URADIN B         55         COR           WHORD I         CORA         URADIN B         55         COR           WHORD I         CORA         URADIN B         15         COR           WHORD I         COR			and white the	1800	2690	1734	371	(mgth)mj	Factor	Eram	lype	455	[william]
WHOST         CARA         LEAD-RIM         E.E.         G09           WHOST         CARA<		6			W24x76	Header	13434216	1115		W21×73	Header	Livorg16.	115
WIGG11         Cock         LEADING         ES         0.00           WIGG11         Cock         <	e.				W24s76	Header	13x3x5/16	388		40° Sinfer	Header	Na Information Available	
WHOST         Cock         HADDRING         ES         COS           WHOST         Cock         LADDRING		-	0		Wiserso	Header	Selection .	18		19-94	Header	14/04/716	z,
Wild III         Cock         Leb/200         15         0.00           Wild III	91		9		W24xE8	Header	Ux24576	115		40' Siefer	Header	12×2×5/16	23.5
WIRDLE         CORP.         CORP. <t< td=""><td>æ</td><td>36</td><td>*</td><td>2</td><td>W24x68</td><td>Header</td><td>03/24/20</td><td>11.5</td><td></td><td>40° Grates</td><td>Header.</td><td>13/2/6/16</td><td>23.5</td></t<>	æ	36	*	2	W24x68	Header	03/24/20	11.5		40° Grates	Header.	13/2/6/16	23.5
	0	43)	1		-			*		*		4.5	*
Windcall         Code         Changes         8.5         0.8           Windcall	2	2	ĕ	2	W24nEB	Header	Living're	115		40' Grider	Header	LNAMETE	20.5
WIRDST         CORN (CORN (C	×	*	9	+	+				÷	+		×	
WINDOLS         CORP.         URADAR         EST         CORR           WINDOLS         CORR         CURDAR         EST         CORR           WINDOLS         CORR         CURDAR         EST         CORR           WINDOLS         CORR         CURDAR         EST         CORR           WINDOLS         CORR         EST         CORR         CORR           WINDOLS         CORR         ES	P.O.	P. 1		,	-	Header	21/24/216	118		40' Goder	Header	DOM:16	21.5
WRIGHT         Code         CRADER         65         0.00           WRIGHT         Code         LEADAR         8.5         0.00           WRIGHT         Code <t< td=""><td>×</td><td></td><td>÷</td><td>35</td><td></td><td>Heider</td><td>Usasyté</td><td>115</td><td>3.</td><td>40' Garber</td><td>Header</td><td>Lb/b/g16</td><td>20.5</td></t<>	×		÷	35		Heider	Usasyté	115	3.	40' Garber	Header	Lb/b/g16	20.5
WR021         Cock         Leb208         85         08           WR022         Cock         Leb208         85         08           WR023         Cock         Leb208         85         08           WR024         Cock         Leb208         85         08 </td <td></td> <td>×</td> <td>-</td> <td>10</td> <td></td> <td>Header</td> <td>905-60</td> <td>115</td> <td></td> <td>40' Geber</td> <td>Header</td> <td>315/047</td> <td>27.5</td>		×	-	10		Header	905-60	115		40' Geber	Header	315/047	27.5
WHORT 1         Cock 1         LEAD 200         15.         COST           WHORT 1         Cock 2         LEAD 200         15.         COST           WHORT 1         Cock 3         LEAD 200         15.         COST           WHORT 1         Cock 4         LEAD 200         15.         COST           WHORT 1         Cock 4         LEAD 200         15.         COST           WHORT 1         Cock 4         LEAD 200         15.         COST           WHORT 2         Cock 4         LEAD 200         15.         COST           WHORT 3         Cock 4         LEAD 200         15.         COST						Header	915000	115		40' Gades	Header	0.04916	20.5
WHOST 1         CORP         READING         EST         CORP           WHOST 1         CORP         LEADING         EST         CORP           WHOST 1	×	(8)	4			Header	0.00476	11.5	s	40° Girder	Header	13x3x5/16	20.5
WIRDST         CORP         LEAD-2019         E.E.         CORP           WIRDST         CORP         LEAD-2019         E.E.         CORP           WIRDST         CORP         E.E.         CORP         CORP           WIRDST         CORP         E.		×				Header	0.004916	11.5		40° Grahm	Header	Usasgre	23.5
WHEAT         Cohe         LEADING         8.5         COR           WHEAT         Cohe         LEADING	2.81					Header	13/05/16	115		40° Girder	Header	Lhosgie	20.5
WIRDID         GORDA         LEADARD         ES         CORR           WIRDIT         GORD	*	æ	÷	.0	W24x68	Heater	13/06/16	11.5	*	40° Grafter	Header	01/04/01/0	22.5
WHO   Cock   CADADA   ES   COS				,		Header	13:04:216	118		40' Guder	Heador	Lhoig16	215
WRIGHT         Cock         URDARD         8.5         0.00           WRIGHT         Cock         <		4	*			Header	13/0/2/16	115		40° Grafter	Header	Lavanghe	20.5
WIRDLY         Cock         DEADLY         ES         0.00           WIRDLY         Cock         DEADLY				-	W24s59	Header	Linksyte	115		40° Greiter	Header	Dongtie	3112
WHOST 1         Cock         DEAD 2019         ES         COST           WHOST 2         Cock         DEAD 2019         ES         COST           WHOST 2         Cock         DEAD 2019         ES         COST           WHOST 2         Cock         DEAD 2019         ES         COST           WHOST 3         Cock         DEAD 2019         ES         COST <t< td=""><td></td><td>9</td><td></td><td>,</td><td>W24s68</td><td>Header</td><td>13x3x5/16</td><td>115</td><td>,</td><td>40' Siefer</td><td>Header</td><td>13/04/216</td><td>23.5</td></t<>		9		,	W24s68	Header	13x3x5/16	115	,	40' Siefer	Header	13/04/216	23.5
WHO   Cock						Header	Living/16	115		40' Girber	Heafer	13/3/5/16	23.5
WHO   Coh   Coh   WHO   Coh	- 0	(-)	÷			Header	DASH	118		40° Grater	Header	Lhoidte	202
WRIGHT         Cocks         URADNIS         8.6         0.6           WRIGHT         Cocks         URADNIS         8.6         0.6           WRIGHT         Cocks         URADNIS         8.6         0.6           WRIGHT         Cocks         URADNIS         8.6         0.0           WRIGHT         Cock	2	œ		2		Header	13×2/16	115		40° Grifer	Header	Lindsofte	20.5
WHOLES         CACA INTERNATION	8	20	e		-	Header	0.05676	311		40° Girder	Header	0.09610	23.5
WHOON         CARA         LEARNING         27.5         2.0           WHOON         CARA         LEARNING         27.5         2.0           WHOON         CARA         LEARNING         2.6         0.0           WHOON         CARA		E	E	,	-	Header	13/29/216	23.5		W19c71	Header	Updag16	118
WINGST         Gode         LEADING         15         CORD           WINGST         Gode         <	ı	Œ	74			Header	13x3x5/16	212	:	W33×130	Header	L3x295g16.	22.5
WIRGS1         Golds         LEADING         \$1.5         CORR           WIRDS1         Golds         LEADING         \$1.5         CORR           WIRDS2         Golds         LEADING         \$1.5         CORR           WIRDS3         Golds         LEADING         \$1.5         CORR           WIRDS4         Golds         LEADING         \$1.5         CORR <t< td=""><td>10</td><td>*1</td><td>60</td><td></td><td>-</td><td>Header</td><td>01/24/216</td><td>23.5</td><td>*</td><td>W24x62</td><td>Header</td><td>13/09/216</td><td>11.5</td></t<>	10	*1	60		-	Header	01/24/216	23.5	*	W24x62	Header	13/09/216	11.5
WHOSE         CICH (EADLING         ELS (EADLING         ELS (EADLING         CORRESS           WHOSE (CICH (EADLING         ELS (EADLING         <	20		31			Header	13×3×6/16	22.5		W24x62	Header	DOMES	11.5
WIRDS         Cock         LEADIN         \$1         0.98           WIRDS         Cock         LEADIN         \$2         0.98           WIRDS         Cock         LEADIN         \$2         0.99           WIRDS         Cock         LEADIN         \$1         0.99           WIRDS         Cock         LEADIN         \$1 </td <td></td> <td></td> <td></td> <td>+</td> <td></td> <td>Header</td> <td>51,596,10</td> <td>235</td> <td></td> <td>W24sr62</td> <td>Header</td> <td>Uk34516</td> <td>11.5</td>				+		Header	51,596,10	235		W24sr62	Header	Uk34516	11.5
WINDS         Grah.         LEADING         8.5         0.93           WINDSS	9	*			-	Header	13/29/216	215		W24s62	Header	Undelptig	11.5
WHO	,			+	40' Griber	Header	DOMEN	202		W24e62	Heady	DOM: 16	211
PRODEST         CARA         URADARD         8.4.1         CODE           PRODEST         CARA         URADARD         8.5.         CODB           <					H	Header	Display in	23.6		Wheels	Header	Llydetti.	418
Windows   Golden   Lead-2008   15   15   15   15   15   15   15   1				t	H	- Inde	13-0-676	344		and and	Marke	1309636	311
WHOST CAR         CARA         CRADIN         14.1         CDD           WHOST CAR         CARA         CARADAR         18.1         CDD           WHOST CARA         CARADAR         18.1         CDD           WHOST CARA         CARADAR         18.5         CDD           WHOST CARA         CARADAR         18.5         CDD           WHOST CARA         CARADAR         18.5         CDD           WHOST CARADAR         CARADAR         18.5         CDD				t	-	Hander	Change	218		Wither	Header	DOMONE.	THE .
Windows         Code         Lebolity         1.5         0.09           Windows         Code         Lebolity         8.5         0.09           Windows         Code         Code         0.00           Windows         Lebolity         8.5         0.09           Windows         Code         Code         0.00           Windows         Code         0.00         0.00           Windows				t	-	Header	0.00476	23.5		W34x62	Header	13/2/6/16	H
Wridel         Coh         Medicin         15.         0.09           Wridel         Coh         Coh         0.09         0.09           Wridel         Coh         0.09         0.09			9			Haufer	UND46/16	302	2	W24x62	Header	13/24/218	115
Wingles         Gode         Lebalish         15         038           Wingles         Fare Company         27         1         1           Wingles         Fare Company         23         1         1				t	H	Header	13/34/16	22.5		W24n62	Header	DOMESTE	11.5
Windows   Code   Leachillo   Ed.   Code   Windows   Ed.   Code   Code   Code   Ed.   Code	2	100		H		Header	Down	205		W24n62	Header	Dobages	511
Web09 Scient Next 85 059 Web09 from Rexistra	8		*	00	40° Girder	Header	D/04676	22.5	(3)	W24x62	Heater	13/0/616	11.5
WASTO Tree-Cost WASTO Tree-Cos		1.0				Header	13/24/216	22.2		W24n62	Header	Undergre	11.5
W240'0 Fin R4254'0 21 87 W240'0 Fin R4254'0 71 87 87 87 87 87 87 87 87 87 87 87 87 87	Contra	14c3/30	88	0.59		Hesder	13/39/216	502	2.5	W36x210	Healtr	FONESS	5.
WOACH Trus On	1	#33	10			Header	12500038	n.		W26x210	Trusts Crox	85	+:
				+	W336201	fork	(4ch:172	×	1.78	W39x300	Truss Ons		
W24c76	. 13	10-2-10	, 3		- margaret	- Park	1445-18		- 0.00	With the	Trusts Cres	e e	×
· · · · · · · · · · · · · · · · · · ·	200	CALALIE	ů.	+	WINGS.	TOTAL STREET	Stratus	6	0.00	M JOSEPH	man Oa		0
14.000 W24676 Fin-Hebyel R2352/H 18	*	+	÷	*	Ť	+	*	+		MG-29	Trans On	†	+

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COLUMN 76
YPE CATALOGUE - (
CONNECTION T
INTERIOR COLUMN

			North					South					Sert					West		
Column	Seam	- Libe	204	Sempth Drift	Factor	Bram	Type	5	Limphibing	Factor?	Sean	Type	5.	[within]	Factor?	Bram	fype	155	[w] quban	Factor
Wredio	W22/84	Header (Diffeet)	(40)4536	14.5		W2160	Snt	CAC PURIS	115	92.0	W36x360	Header	L6x2x5216 bent	365		W22584	Header	14x3x576	2.0	53
0014700	9014630	Header (Offset)	15/04/216	45	-	W14622	Code	1442430	3.5	0.58	WOSKIDS	Header	L4x2x5/16 bent	215	æ	W24x62	Header	13/04/216	343	3
	W36k210	Header (Offset)	C4x3x3/B	14.5		W24655	tok	Lévicité	345	100	WRENSO	Header	(4e8/3/8 Sept	55	- 00	W26k210	Header	14e3e32	302	•
901.791.00	9	9		91	į.	3914622	Colt	Lechilli	9.8	0.59	WESTE	Snite	1464638	22.5	162	W24x76	Keste	1444033	14.5	1.00
	36	36	***	3		W14/22	Coulty.	140300	8.5	0.68	WESTE	Cody	C4+4×3/9	23.5	162	W24s76	Colfe	1444433	342	100
W34c133	4)	67	ti	4.	63			0	•	**		-	5)		•			4.5		*
	Q.	æ	æ	j	2	W14c22	Knife	14/3/30	8.5	0.55	WDGKDE	Coulte	C4x4x3/B	20.5	162	W24s76	Code	L4c3c399	5#6	1.00
10146/007	Æ	*	*	T	26	10	1	*	t	8		1.0	*	ť	*	ı	*		ж	15
and the same	100	0.0	163		-	W14c22	Code	14/9/38	8.5	929	Wiecijs	Coult	L4x4x3/S	215	1.62	W24976	Knite	1443438	14.5	100
Meaning	(e)	æ		00	28	W14622	Crafe	L4c2x310	8.5	0.58	WJGKTJS	KNA	1444539	23.5	1.62	W24s76	156.95	14c3c39	516	100
11 (40.01)	16	e		1	25	W14622	Cufe	16/3/38	8.5	0.58	WHENTE	Enth	244433	20.5	1.62	W24s76	Colfe	140009	. 514	1,00
1000000		1		,		W14622	Cale	18/0/20	98	620	261/96W	Carle	Detection	38	1 66	W24676	South	LAGGS	516	100
W148370	2.	4		3.	3.	W14422	Code	(40303)	8.5	650	WIGNIE	Code	[4+4c3]9	×	1.66	W24s76	Kolk	LAcholli	511	1.00
and the same	3	×				W14422	Code	(Actol)	818	0.59	WIGHTS	Epithe .	Likeliji	215	162	W24s76	Knih	(4c)(3))	14.5	100
W348420		×		,	×	W14s22	Coff	(techn)	8.5	0.59	WIGHTE	Knife	Contract	202	180	W24k76	Colfe	DACAGE	14.5	100
100000		÷		ě		W14s22	Code	0(14791)	8.5	0.50	WD6xD5	Knite	(4c4c)()	20.5	1.62	W24s76	Kath	144439	14.5	1.00
WINNESS		1.				W14x22	Crafe	(lectrol)	8.5	650	WJ6x135	Coult	(4c4x)))	23.5	162	W24e76	Code	(Antech)	14.5	100
003-4180						W14622	Cole	\$16,0,031B	20.00	0.59	WIGHTE	Korte	Lévisig	225	1.62	W24s76	Cale	Leekcylli	311	100
0000000				,		W14/22	fok	16/3/38	85	950	Wzertz	finite	(deský))	205	162	W24x76	Keile	1444039	14.5	1.00
Misserro	9					W14s22	Cofe	Lechalli	98	0.59	W36x35	Snife	L4s4c3/B	20.5	162	W24x76	Knife	Leecht.	14.5	1.00
	(1)	*	(8)	*		W14622	Crite	Lechalls	20	0.58	Wasetas	Knite	Lever3/8	23.5	1.62	WZ4678	Keile	LAver39	14.5	100
ON SAMERIC	•5			1.		W14s22	Code	(463038	9.2	950	WHENTE	Courte	(4eks)(8	202	182	W24s76	Kai he	(Arth:39	345	100
	in.	0	er.		0	W14622	KNS.	14-2-38	8.5	0.58	WHEETH	Cook	Cdeda 3/8	502	162	W24n76	Confe	14544.379	5#4	1.00
Witedoor	Œ	Œ,		£	80	W14x22	Krafe	145/21/20	9.2	0.59	WORLDS	Coope	54444.3/0	22.5	1.62	W24s76	Kesh	L4x4x3/9	348	1.00
M INNOCO	3654655	Seit.	Lévicité	118	626	9914422	Code	(4c2c3t)	9.8	0.58	WORLDS	Contr	Lénes/3/9	22.5	162	40° Geder	Stated			
00.29530	924455	Sub-	L4x3x378	11.8	0.29	W21560	Crote	Like Shights	11.5	62.0	W36ctS0	Knite	C4e4x3/B	23.5	162	40' Grifer	Sented			ð
	WASS	Colfe	24x3x338	11.5	0.78	W21450	Cafe	1403030	311	0.78	W36K550	Kpuft.	(4444.3)	20.5	1.62	40' Geriter	Sente	*0	*	*
W14x720 + 2 WEB PI		Kn8	Like he hills	11.5	620	W21450	En.fr	1442438	11.5	629	W36x50	Kesh	1444439	22.5	160	40° Sinter	States			*
14.001	4	£10	(4x3x3/3	115	6.03	W21x50	Crists	16/3/38	11.5	628	W35x135	Knite	(4c4c3)B	20.5	200	40° Gerffer	Tab.	٠	+	*
W144730 + 2 WEB PI		Ku3	[4x3x3])	11.5	673	W21x50	fut.	(4.5.0)	11.0	0.78	WJGKIJS	Cuch	(4e4c))))	212	201	40' Gades	Seated.	***		5
The state of the s	250000	SAR.	Section	11.5	0.79	WZIESO	And a	16.0.00	200	828	WHENTE	Lorie	Cheering Co.	225	201	40.0000	Septem	+		1
W146130 + 2 WEB P2 12.5xd		Cole	Larzellis	11.0	0.79	W2160	Code	Lechallin	118	0.78	WJ6x136	Cole	(Artex)	23.5	162	40° Grefer	Seated			1
What you a supplement	L	Cult	(4c3c30)	11.5	620	W2160	Cute	(46.25.37)	311	0.78	W36x35	Coult	(4c)c30	z	100	40° Gorden	Seated			1
1254		Knife	LACICIE	11.6	620	W2169	fink	16/3/38	115	620	WJG-13E	Korfe	MONT	K	1.00	40' Gağer	Stated			,
W14r320+216E	PR W24655	Sisk	LACASAR	11.5	620	W2560	Cale	(4e2e3)))	11.5	67.0	WIGHTE	South	LACKUR	A	1.66	40° Grider	System			1
12.54	W24455	Sufe.	(4c)c)())	911	67.0	W21450	Colfe	Lech300	311	67.0	W30c135	Code	(4c0:39)	25	1,00	40° Girdes	Seated	20.	20	20
W14c730 + 2 WEB PI	P W24455	Code	Ukchollh.	11.5	0.79	W21x50	Code	(4004))	11.5	0.78	WIGHTE	Coult	(4c2c3)	25	1,66	40° Goder	Seated	6	8	*
12.566	W24455	Krife	C4c2c1/8	116	620	W21450	Krish	(6/3/3))	11.6	64.0	WHEELD	Cook	54cb30	A	180	40' Grifer	Seated	500		
W34x720 + 2 WEB PI	PI W24455	Krish.	54:2139	11.5	0.79	W21450	Knite	16(2)(0)	415	0.79	WHOLDS	Enth	54c3c30	×	1.00	40' Girfer	Seated	20	20	2
12.568	W24455	Sub.	LACACITI	118	0.79	W21450	Code	(442438)	111.5	0.79	WHENTE	Corte	LArZc2B	2	1.06	40° Gerber	Sented		14.7	
Sits on Truss 1	3	38		œ.	N.	34	35	336	÷	æ	e.	35	50.5	33	×		3	3.5	31	3.5
810	+3	+0	F-3	9:3	+3	+		+3	+:	*		V.	*	70	٠	v	10	100	+	+1
9				3						÷										*
x		æ	æ	o .	18	œ	æ	36	3	OK.	98	w	38	*	÷	*	:	38	96	i.t
	40	*	÷	1	6	0	6	200	£	50	15	50	65	12			- 13		9	*

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity
Where no beams size is indicated no member is framing into the column
The Information specified for each connection was taken from the latest corresponding frankel Steel Limited steel shop drawing for that connection

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			750.00					10000										Well		
Column	Beam	- Inpe	100	length (in)	Factor!	Beam	Type	ž	Limpth Drill	Factor?	Beam	Type	5.	Inthem.	Factor?	Bram	fype	155	[w] quban	Factor
Wredo	W21-60	Code	LACASIN	8.5	650	W14x22	de S	164,9438	88	650	W3069	Heider	L4x2x5/16 bent	14.5		WZZNSK	Healer	Ukdbd716 best	25.55	1
New Action	Williams	Coste	(4x3x3/0)	14.5	100	W16d1	Code	1442930	8.5	0.58	W23x84	Header	LSQM216 best-14s3x5/16 best	382	128	W24s76	Healtr	L4cOrd/16 bent	345	88
O CONTRACTOR OF THE PARTY OF TH	W24455	Kn%	E4x3x3/B	9.8	659	W14422	fish.	1643439	8.5	850	W22r84	Header	15x3x5/16 best-14x3x5/16 best	596	- 02	W2754	Header	(40)6216 best	511	*
1000000	W14622	Knik	L4x3x3/III	8.6	650	1014422	Kole	LACKUR	9.8	0.59	W27/84	Header	L5:045/16 best-(4s/342/16 best	.5#4		W24x68	Header	LExb4/16 bene-Ukrah4/16 bent	445	
10000	W14/22	Colfe	L4x3x3/III	8.5	650	W14/22	Cooks	1405/33	8.5	0.59	W22684	Header	LSc2e5/16 best-14x3hf/16 best	34.5	æ	W24st/8	Heafer	(Sx3xg/16 bene-14x3x5/16 bent	342	*
OR SALVED	-	4			42	5	0	43	+	**				-			*	**		*
2134120	W14622	KN5-	E4x3x3/8	9.8	659	W14/22	Code	\$6CH300	8.5	0.58	W27:64	Headts	LEXTHS/16 beat-14r3ts/16 best	514		W24s68	Header	LSchights bene-Life 2h5/16 bent	345	2
1000-000	f	+		r	:6	*		+	÷	8		1		Ŧ	*	ı	*			*
Carrant	W14422	Chile	L4x3x33	8.5	950	W14-02	Code	14/3/33	88	920	W22k84	Header	L5s245/16 best-14s245/16 best	345		W24x68	Healer	15/04/16 best 44/04/16 best	345	2
10.4	1014622	KssNe	L4x3x3/B	98	650	W14622	Colle	(4c5x)))	8.6	0.58	W22rd4	Header	L5x2x5/16 best-[4x3x5/16 best	316		W24s68	Header	LSichs/16 bene-L443KS/16 bent	311	8.
14740170	W14/22	Colfe	(Actal)	8.5	650	W14422	Cish	16000	918	0.59	W27594	Header	15c24576 best-14c24576 best	.516	*	W24e68	Header	USGNEYNS best-Lechtly 10 best	145	*
Meschan	W14/22	Colle	16000	9.8	650	W14622	Cale	16/3/38	8.5	020	W27694	Header	Lkv2x576 bent	14.5		W24e68	Heador	LSGNS718 bene-Like265716 bent	345	3
	W14422	Oute	L4c3c3/B	8.5	650	W14422	Civite	(46.34.)(8	9.8	650	W23x94	Header	L4x245/16 bent	14.5		W24e68	Header	LSGNS716 bere-L4r2HS716 best	145	3.
00.00	W14422	Colt	L4c3c3/B	8.5	650	W14422	Code	160038	115	0.59	W23x84	Header	L4x25/216 bmt	245		W24e68	Header	LExital Ni bent-Lite 245/16 bent	14.5	5
	W14422	Knife	Ue2d/8	8.6	650	W14/22	Chile	56/3/38	8.5	650	W27s54	Header	Lechs/16 best	14.5		Wateria	Header	LSchig16 bene-Likr2HS/16 bent	145	3
tara-hea	W14/22	Crafe	Uk/3kJ/B	9.0	050	W14422	Kriste	00000	9.2	050	W23id4	Heater	L4x2x5/16 bent	395	*	W24x68	Heady	LEXIDS/16 bers: L4x2x5/16 bert	145	*
THE REAL PROPERTY.	W14/22	Colfe	LACKSIN	8.6	650	W14s22	Code	(16/3/3)	88	0.59	W23x54	Header	L4x2x5216 bent	14.5		W24n68	Heador	USchill 16 bert-Like Dis216 bent	345	
0007790	W14/22	Coste	LW3x3/B	9.8	650	W14622	Colfe	9(6)(3)(3)	10 10	0.59	W27594	Header	14x245/16 bmt	345	*	W24x68	Heador	(5:015)16 best-(4:315)16 best	346	3
10/63/10	W14G2	Code	EN-BA3B.	98	659	W14/22	foh	16/3/30	8.5	950	W2764	Header	Lkx2x5716 bmrt	391		W24e68	Header	USGNS/16 been Leichs/16 bent	14.5	*
And Annual	W14/22	Knik	teach	. 99	650	W14s22	Kah	Lechtiff	98	0.59	W27x84	Header	14x3x6716 bent	14.5	,	W24x68	Header	LSchifflübere-Litezhiffühert	3#2	1
	W19422	CHR	L4x3x3/8	8.5	650	W14/22	Crete	160338	8.5	0.58	W27x94	Header	L4x3x5/16 bent	14.5		WZ4w68	Header	L5ch5/16 best-Uk315/16 best	14.5	×
200	1814422	Code	1443438	88	450	W14e22	Colt	16008	8.5	950	W27484	Header	L4x3x5/16 bmit	545		W24s56	Healer	USahS/36 best-UsahS/36 best	345	*
1000000	W14x22	KNE	E4r2r3/8	8.5	450	W14/22	Colfe	\$6.54393	8.5	0.59	W23584	Header	14x245/16 temt	5#4		W24+69	Heafer	LSchs/16 benc. LAr2nS/16 bent	5#4	20
337-75.00	7018/22	Kinde.	L4x3x3/III	4.5	650	W14s22	Krish	16/2/30	38	0.59	W23/84	Header	Lexibilità best	14.5	(4)	W24H/B	Header	L5x3x5/16 bens-L4x3x5/16 bent	345	2.
	1914/22	Knite	Lécholité	88	650	W14422	top	(NCA)	58	0.08	W23584	Header	L4x2x6216 bent	14.5		W3069	Healer	Uschell'16 bent-Lechtlift bent	23.5	1
003494000	W2160	Code	L4x2x3/B	11.5	629	W16G1	Crafe	Lechale	8.5	62.0	WD089	Header	14x2x5/16 best	14.5	æ	W27x84	Header	Linkid (8 best-Like Ind) 8 best	22.5	ं
	W21450	Corit	£4x3x3/B	11.6	620	W10.01	Confe	563030	8.5	0.58	W2049	Header	(4x2nf/16 bmt	345	*	W27584	Healer	15/215/16 Seet-L4/215/10 Sent	14.5	20
(MYANGE)	W23450	Kn3s	Like he this	11.5	62.0	W16/31	Sn5	(Redolf)	9.6	0.69	WD069	Header	USCANTE best-14424516 best	14.5	.0	W27x84	Header	USchol/16 bens-Like 2ng/16 bent	345	
	W23450	Coste	(4x3x3/8)	115	67.0	W1601	Crite	16039	8.5	6.59	W23584	Header	L5x3x5/16 best-L4x3x5/16 best	14.5	٠	W27x84	Header	LSchill 16 best-Like 245/16 best	345	18
W14x605	W21x50	Code	L4x3x3/8	11.5	640	W1601	Kn3	Leanth Leanth	8.6	0.58	W23x84	Header	LSuzhighti bent-14rzhighti best	2.5	*	W27x84	Heado	L5x2x5/16 bent-14x3x5/16 bent	## ##	5
No.	W21450	Colb	[4x2x1/8]	511	679	W1601	Code	56-2-13	9.8	0.58	W23/84	Header	L5c2rG16 best-[4c2rG16 best	382	7	W27584	Heady	LSchiff Sest-(4chiff Sept	345	1
W14x605	W23-60	Knft	(4c2c38	11.5	679	W1901	Keste	14c/x38	98	0.59	W22d4	Header	LSchigft best-LAchsylf best	14.5		W27x64	Header	LSGPG/16 bero L4r2hS/16 bert	345	*
	W2 te50	Chille	LACACITE	11.00	0.29	W16-01	en en	(40.00)	8.5	0.59	W22584	Header	LSGPS718 bent-(4r.DrS716 bent	14.5		W27584	Header	USkitely 18 bene-L4k 205/16 bent	14.5	1
W14x1665	W25450	Colt	Ueballi.	11.5	0.78	W16d1	Coh	(40.70)	982	000	W23/84	Header	LSGPG16 best-14chef/16 bett	345	2	W27984	Heady	Usubly 16 bere-14-24/16 bert	318	2
+	(S017M	MON	000000	11.0	670	WIGHT	ASS.	BENEVAL)	0.0	800	*0.00M	negative.	ESCRIPTION OF THE PROPERTY OF	0 2	-	100,000	and and	TOTAL STREET,	0	
W14m505	W21460	Confe	LACKER	115	679	W16.01	Calle	Lechalli	8.6	0.00	W23d4	Header	Uschell Chest-14-26/16 bert	14.5	2	W27x84	linater	Update/16 bere-like and/16 bere	345	2
NAME OF THE PARTY	W21450	Code	UAraram	11.5	0.79	Witech	Code	160000	8.5	0.08	W23x54	Header	Updat/16 best-(4c2ct/16 best	14.5		W27x64	Healer	Updat/76 bero (4c2st/16 bert	345	*
W14x730	W21450	Colfe	(4c2c1))	115	420	WIGGT	Knite	(6.0.0)))	85	95.0	W27c94	Header	LEXINGTE best-14chtf/16 best	514		W22584	Heady	LSch@18 besc.l4ch@1676 best	514	12
A100 - 1000	W21450	Knife	U45343B	11.5	0.79	W10.01	Knite	00000	4.5	0.59	W23/04	Header	LScarg16 best-lAr2eS16 best	14.5	0	W27x64	Heady	USGNETS benchtschift bent	345	2.
No. of the last of	WITHOU	Colle	LACASTR	11.6	626	10014	tes	(46.20.31)	9.8	0.59	WZZNB4	Header	USchight best-likrbright best	145		W27494	Heater	USONSTREEME (REDISTREEM	14.5	1
Sits on Truss	1.2				115				Č+	35	e e	35			25	35		3.5	21	33
15	+0	+	*6	10	+3	+		-	16	*		4		90	*1	+1	*	80	+3	(4)
34	.0				0.					÷									0.	
1	*	æ	æ	or.	18	æ	æ	99	9	æ	ě	ě	38	*		÷	÷	æ	96	ð.
ŧ.	6	6	to	10	6	6	6	22	÷	50		5	60	*		5	5	9	9	*

I state factor instell for each fair confection types as they are conservable and state of the confection of the confection types as they are conservable y samed to have sufficient tensile capacity.

Where no beams size is indicated no member is framing into the column.

The Information specified for each connection was taken from the latest corresponding frankel Steel Limited steel shop drawing for that connection.

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PPE CATALOGUE - COLUMN 78	
INTERIOR COLUMN CONNECTION TY	
C5.21	

1   1   1   1   1   1   1   1   1   1	Marie   150,   Mari																					
1.   1.   1.   1.   1.   1.   1.   1.	Melber         URDARDIN         11.5         N GRANG         Instant         URDARDIN           Instant         URDARDIN         11.5         N GRANG         Instant         URDARDIN           Instant         URDARDIN         14.5         N KAREN         Instant         URDARDIN           Instant         URDARDIN         44.5         N KARENDA         Instant         URDARDIN           Instant         URDARDIN         44.5         N KARENDA         URDARDIN         URDARDIN           Instant         URDARDIN         44.5         N KARENDA         URDARDIN         URDARDIN           Instant         URDARDIN         URDARDIN         URDARDIN         URDARDIN         URDARDIN	Column	Stein	1104	No.	Sempliful (m)	Factor	Besen	Type	5.	Limpth Srd	Factor?	Beam	Type	5.	[w] qubus	Factor?	Eram	fype	204	[w] quban	Factor
Mail	Missilamin         Diabelity         18.5         n. 10-bid/18           Missilamin         Diabelity         18.5         n. 10-bid/18         memor         Diabelity           Missilamin         Diabelity         18.5         n. 10-bid/18         memor         Diabelity           Missilamin         Diabelity         18.5         n. 10-bid/18         memor         Diabelity           Missilamin         Missilamin         18.5         n. 10-bid/18         memor         Diabelity           Missilamin         Diabelity         18.5         n. 10-bid/18         memor         Dia	W14d0	W14422	Code	LACALIB	88	0.58	W14x22	Code	Chr. Jan 18	88	0.59	W24still	Header	13/246/16	115		W24x76	Header	Lborg16	115	
1	Header   Clondoffs   NS 5	Widelin	W16-21	Colfe	(4x3x3/8)	2.0	650	W14622	Code	1443439	3.5	0.59	W24x84	Header	01/54040	586	æ	W24s76	Header	13:04916	382	
1	Healer   Clandoff   N. 15   Victoria   Healer   Clandoff   Victoria		W14/22	Sub.	S4x3x3/B	311	0.79	W21s44	Keh.	(6c)c)()	8.5	950	W22c130	Header	13/29/216	20.5	20	W26k150	Header	Chongité	20.5	
1   1   1   1   1   1   1   1   1   1	Healer   Linodyty   N.S   Workel   Healer   Linodyty	Whatest	W14422	Colle	54c3c3/8	88	650	W16c26	Colfe	Lechilli	9.5	0.59	W27x84	Header	Dx265/16	14.5	ď	W24x68	Header	13×2×5/16	115	.1
1.00   1.00	Heater		W14422	Knife	14×3×3/8	8.5	650	W16/26	Code	14c3x30	8.5	0.63	W23x84	Header	Dio6/16	34.5	æ	W24n/ib	Header	13x3x6716	115	*
14.00   1.00	Heatin   Nichtment Analysis   1, 10 ct   10	OKA9930	43	43	10	4.0	43	-	-	63	6			-	•		-	*		- 53	* 3	*
WYORD                 MATERIAL STATES                 CALLES AND ART	Heater		1974422	KNS	£4x3x1/8	8.5	659	Wt626	K649	\$45.00 M	8.5	0.68	W22584	Header	His Information Available	Ť	83	W24s68	Header	Livingte	511	7
Ministry   Ministry	Hoteler   Nich Merstand Analysis   1, 100-05/14	Wide SEC	+1	*	t	r	26	ж		*	÷	8			+	Œ	æ		+ 5		ж	
Ministry   Color   C	Hoteler   No Internation Analysis   No Section   No Sec	1000000	W14422	Code	CAr Sealins	8.5	650	W16425	Code	1443438	8.5	928	W27484	Header	No information Available	1	10	W24668	Healer	0.04916	115	
Mile	Helpine   Nicholated Aucholes   Nicholated Auchole   Nicholated Aucholated Auchole   Nicholated Aucholated Aucholate	Mrs. with	1014422	Cult	L4x2x3/8	98	650	W16626	699	L0c3x310	8.5	0.50	W22x8A	Heider	No Information Available			W24x68	Healer	UN24916	115	8.
Ministry   Graph   G	Healer         Libody(N)         N.S.         N/CRARD         Insular         Diody(N)           Healer         Libody(N)         N.S.         N/CRARD         Insular         Diody(N)           Healer         Libody(N)         N.S.         N/CRARD         Healer         Libody(N)           Healer<	10124610	W14/22	Colfe	MERS)	8.5	650	W1626	Code	16/3/33	918	0.59	W27984	Header	No information Available	×	35	W24e68	Header	DOMEST	11.5	
	Hologope	Mark Sans	W14422	Code	DADAIR	98	650	W16/26	Cale	160,030	8.5	0.59	W27x8A	Header	0.000.0	14.5		W24x68	Heador	100416	115	
Mile	Healer   Libody   M. S.   Workey   Healer   Libody     Healer   Libody   M. S.   Workey   Healer   Libody     Healer   Libody   M. S.   Workey   Healer     Healer   Libody   M. S.   Workey     Healer   Lib	0.146233	W14422	Cult	(4c)c)(I	3.6	650	W16/26	Cult	(4chill)	8.5	650	W23x84	Header	31/5/04(1)	14.5		W24e88	Healer	13/2/6/16	11.5	
1   1   1   1   1   1   1   1   1   1	House	The same	W14422	Colt	L4x2x3/B	8.5	650	W16/26	Code	160038	8.5	0.50	W23/484	Header	DOGNE	145		WOARD	Healer	Undelgtik	11.5	
WHY                 WAY                 NAME                 WAY                 NAME                 MAY                NAME                 MAY                 NAME                 MAY                 NAME                 MAY                 NAME                 MAY                 NAME                 NAME                 NAME                 MAY                 NAME                 NAME                 NAME                 NAME                 NAME                  NAME                  NAME                  NAME                  NAME                  NAME                  NAME                       NAME                       NAME                      NAME                       NAME                      NAME                       NAME                       NAME                       NAME                       NAME                       NAME                       NAME	Holorida   13, 15	N34680	W14.22	Kuite	UpOd/8	98	650	W16/26	Dafe	(technille	8.6	0.59	W27s84	Header	31/50/11	14.5	:	Water	Header	Undergite.	115	3
Minical   Mini	Helder	CONTRACTOR	W14622	- Khih	Ltv2v30	9.8	650	W16/26	Code	0(1/2/20)	8.5	050	W23x84	Healer	13/06/16	145	180	W24w58	Headyr	Didig16	115	*
WHY                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                  CARS                 CARS                  CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                 CARS                  CARS                 CARS                  CARS                  CARS                   CARS                      CARS                   CARS                  CARS                   CARS                  CARS                   CARS                      CARS                   CARS                  CARS                   CARS                  CARS                   CARS                  CARS                   CARS                  CARS                   CARS                       CARS                        CARS                   CARS                   CARS	Hacker   Diodoffs	W16c362	WH4422	Colfe	LACICIE	98	650	W16/26	Code	Lechalli	8.5	650	W27184	Header	13-0-676	145		W24n68	Healer	LhOng16	115	
Mary   Mary   Graph   Graph	Helper         LDAGONY         NS         TO PROPER           Helper         LDAGONY         NS         TO GREGIS Invent         LDAGON           Helper         LDAGONY         NS         GREGIS Inven	-	W14/22	Code	Decorate	8.5	659	W16/26	Colle	(lechal)	10 00	0.59	W27s84	Header	31/20013	14.5		W24x68	Heador	UNDAGHE	11.5	
WHY ST                 Color                 Lock State                 Color                 Lock State                 Color State                 Color State                Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                 Color State                  Color State                  Color State                  Color State                       Color State                       Color State                      Color State                       Color State                       Color State                       Color State                        Color State                        Color State	Healer   DJOSOFY	0/59444	W14G2	Kult	SN313B	9.6	650	W16/26	foh	16000	8.5	950	W27x84	Header	31/5/010	14.5		Waterielle	Healer	Upagt6	511	
Wind bloom   Wind bloom   Wind   Wi	With the color of the	00144-200	W14/22	Keile	144343/8	8.6	650	W16c26	fah	Lechtliff	98	0.59	W27x84	Header	03/5/20	14.5	,	W24x68	Header	13x3x6716	115	
WHANG         Giolea         Chickaga         Giolea         Giolea         Chickaga         Giolea         Giolea         Chickaga         Giolea         Giol	Healer   Libody   M S		W18422	Knfe	L4x3x3/8	88	0.58	W16/26	Code	Lechsille	50	650	W23/84	Header	13/2/01	14.5	÷	WZ4HEB	Header	Langerte	tts	
Wind blook   Wind blook   Wind   Wi	Hearine   Libody/18   14.5 - 1   10-040/18	MITANATA	1014422	Code	14434378	8.5	650	W16c26	Code	(60038	9.2	950	W27484	Header	Dogs	14.5		W24e56	Healer	Lhospits.	115	
Windley   Windley   Windley   Grie   Color   Windley   Grie   Windley   Grie   Windley   Windley   Windley   Windley   Windley   Grie   Windley   Grie   Windley   Grie   Windley   Windley   Windley   Windley   Windley   Grie   Windley   Grie   Windley   Windley   Windley   Windley   Windley   Windley   Windley   Windley   Windley   Grie   Windley   Win	Heater   Libody   N	NAME OF	W14s22	Knife	\$6.3×3/8	9.8	450	:W16-26	Colfe	\$6434B	8.5	0.69	W27x84	Header	13/949/16	14.5	i.e	W24n68	Header	21/54/47	115	
Wind   Color   Color	Herate	W14s455	W14/22	Kristy	L4x3x3/III	4.6	650	W10/26	Knite	1442430	98	0.50	W23/64	Hesder	0.09670	14.5	8	W24MB	Header	13/04/216	11.5	
Windle   Cock   Cock	Hearles   Chordy   N. 4. 5	1000000	1914/22	Colt	Lévinille	5.0	650	W10/26	Code	(4c)cith	58	950	W23584	Header	13/20/216	12.5		GRDERAG	Heater	Lbcb/g16	23.5	
Windle   Cock   Cock	Heater   Libody   N. 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1,	00346400	W1801	Coste	L4x2x3/B	8.6	620	W16G1	Coste	L4c3x38	8.5	62.0	W23x84	Header	Dx26576	14.5	æ	0903069	Header	13/29/216	23.5	
Windlesson   Win	Nearer		WIGGT	Colfe	C4x.3x378	8.5	650	Wradi	Cafe	14c3c30	9.8	0.58	W27s84	Header	03/5/5/26	14.5	*	0909090	Header	0109610	215	
Windle   Cock   Cock	Heater   Lindoff   N. 5   000880   Heater   Lindoff   Heater   Lindoff   N. 5   000880   Heater   Lindoff   N. 5   0008	WYNESSO	WISGI	Kn%	Like he tilling	8.6	650	W16/31	Cate	1443438	9.5	0.69	W27x84	Header	13434576	14.5	e :	0469690	Header	13/04/16	215	
Windle   Windle   Cock   Cock   Cock   Cock   Cock   Cock   Windle   Cock   Cock   Windle   Cock   Windle   Cock   Windle   Cock   Windle   Cock	Healer   Lindoff   N. 1.   Goldfiel   Healer   Lindoff		WIBGI	Custe	L4x3x3tB	2 80	450	W19626	Ent.	Lecksing.	8.5	850	W23x84	Header	31,596713	14.5	٠	GROENAO	Header	Chronerie	225	
Wind   Color   Color	Herete	WHANGE	W16/21	Cult	[4x3x3/B	40	650	W16/26	Enth.	(44.343)	82	850	W22x84	Header	1306/16	27		09006940	Header	Changing	27.5	
	Heater   Lindoff   N.	No contract of	WIGGI.	Kun.	Section	40	650	WIGGS	Ann.	160.00	6.0	0.50	WZZNIA	Heater	0.000016	58.		CONDENS	Healty	Changing	225	
RYREAD         CICAL CASA         CICAL CASA         CICAL CASA         6.6 kg         CICAL CASA         6.8 kg         CICAL CASA         FIRST         CICAL CASA         CICAL CASA         CICAL CASA         CICAL CASA         FIRST         CICAL CASA         C	Nasier   Diodyl   14.5   0.000   Nasier   Diodyl   Natier   Diodyl   Diod	WHANGE	W16/21	Code	140.0030	0.00	0.00	97954M	Contract	144,418	68	0.00	W23v84	Header	Debeth Charles	277		OMDEDITO	Header	Living He	23.6	
Winding   Winding   George   Lebergia   See   George   Lebergia   See   Control   Co	Number   Diodelfor   N. 1   October   Number   Diodelfor		1010031	Code	(14/4)(1)	3.0	0.63	Wednesd	Code	(fachall)	9.6	0.68	MD 3-64	Header	13/09/01	376	3 2	CHEDEBAO	Healer	1309616	21.6	
Windle   Cach   Cach	Interior   Libodoffs	W14x665	W1601	Colfe	(4/2/3/8	86	0.59	W16/26	Code	(40,038	88	0.69	W23484	Header	0.00016	14.5		онадено	Header	Charges	23.5	L
Windly   Windly   Cock   Coc	Newton   Linochys	2000	W16G1	Colle	1442430	98	650	W16/26	Cale	(4000)	9.5	0.53	W27x84	Header	03/5/07	14.5		0905040	Header	13/0/67/6	23.5	L
Windsty   Windsty   Code   Location   8.1   Code   Windsty   Code   Windsty   Code   Windsty	Hacker   10-000FW	W 1405005	W1603	Code	\$400dh	9.0	0.59	W16/26	Cuffe	16050	8.6	0.09	W23/84	Header	01/2/0/07	14.5	(2)	090000	Header	13/24/210	20.5	
Windle   Windle   Windle   State   Colorado   State   Colorado   State   Windle	Neutrin   Christoffs   M. S.   Goldston   Ch	Carles Comp	W1601	Code	1,4×3×3/8	9.6	650	W16s26	Code	(400)	9.0	0.09	W23x84	Header	31/94210	14.5		GADERA	Healer	DISMITTE	27.5	
Windle   W	Newton	W148730	WHEST	Knife	U4/341/8	8.5	659	W16/26	Knife	((4/2)))	8.5	0.59	W23484	Header	DOMEN	514		GIRDERAG	Header	Lhdugtis	202	
State fractor   Scale   Lebrato   State   Connection analyzed by Ingraffes for axial capacity and Stiffness	Hearter	9614-230	W16.01	Kriste	U453570	4.6	650	W16/26	Colt.	16(3(3))	4.5	0.09	W22x84	Header	13/0/6/16	14.5	(3)	09005940	Header	13/0/616	20.5	
Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	siffness		W16/21	- Code	Lécholiti	s a	650	W16/26	Code	Lechtlife	9.5	0.59	W23484	Header	13/29/216	12.0	5	GRDERett	Heater	Changes	22.5	
Scale factor listed for each knife connection ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	siffness	Sits on Transfe		26		0	33	33	35		i.t	35	e e	æ	35		as.	35	0.5	3.6	35	
Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	stiffness	F	+:	+1	r:	,	+.			+3	ti	*:		ŀ	*	ti	.*:	+	+5	85	+3	
Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	siffness	ų																				
Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial rapacity and stiffness	siffness	a.	æ	9	9	9	8	96	œ	90	3	98	ě	ė	96		÷	*	.ti	×	99	đ
1 Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	1 Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5' deep connection analyzed by Ingraffea for axial capacity and stiffness No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity	1	2		20		6	9	9	22	t	5	5		9	12	***	*		9	9	5)
1 Scale factor listed for each krife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness	1 Scale factor listed for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity	,	17	æ	i.e	9	+	+		æ	it.	12				7	ir.	7		i t	+	
	No factor fisted for other connection types as they are conservatively assumed to have sufficient tensile capacity	1 Scale fac	or listed for	each knife c	connection correspo	nds to ratio	of depth t	o the 14.5	deep conne	ction analyzed by I	ngraffea fo.	axial cap	acity and s	tiffness								

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Mail		Column	Beam	lype	Sate	Length [m]	Factor	Brann	Type	504	Length Dell	Factor	Beam	Type	455	[mgthfe]	Factor	Eram	lype	Sur	Length [n]	Eactor
	1	Wiledly	WHATSO	Header	LAcholds best Lluckolds best	38.5		W2049	Code	Leodis	316	1.06				6		Wolecting	Header PT	943/8	u	Ш
Wind   State	_	W14e132	Waterao	Sented	*			W24:94	Knife.	Lecocite	14.5	1.00		×		×	÷	WEGGE	Header PI	9/0/6	75	Н
Mary	1		WINTED	Seated	*	, .		W22/94	Code	16(3/3)	145	1.00		5	8		5	W799-150	Easte.	85x3/8 best	345	4
WHO         WHO         COLUMN		W14x193	WINTED	Seated	*	,		W27x94	Cole	160038	14.8	100			*	*		WHISTIE	Kelk	850/8 hert	N.K.	4
WHATE         Control	1		Waterass	Seated		,		W27x34	Kriste	technille	145	100				*	*	Washis	Kale	85x3/8 best	37.	4
Fig. 15   Fig.	-	WHERE			*	,																4
Hybrid                 Column                  Column                 Column                 Column                 Column                 Column                 Column                 Column                 Column                 Column                 Column                 Column <t< td=""><td></td><td></td><td>W33x130</td><td>Seated</td><td></td><td></td><td></td><td>W27x84</td><td>Code</td><td>(4ch:28</td><td>34.5</td><td>100</td><td></td><td></td><td></td><td></td><td></td><td>W26x15</td><td>Kali</td><td>B \$43/8 lient</td><td>24.5</td><td>4</td></t<>			W33x130	Seated				W27x84	Code	(4ch:28	34.5	100						W26x15	Kali	B \$43/8 lient	24.5	4
		Wide/III							*		*			*	×	*	-					Н
(1)         (1) <td></td> <td></td> <td>WENTED</td> <td>Seated</td> <td>+3</td> <td></td> <td>0.</td> <td>W27694</td> <td>Code</td> <td>Lecholis</td> <td>14.5</td> <td>1.00</td> <td></td> <td>+</td> <td></td> <td></td> <td></td> <td>WINNER</td> <td>Colte</td> <td>B.Sx3/8 bent</td> <td>345</td> <td>Н</td>			WENTED	Seated	+3		0.	W27694	Code	Lecholis	14.5	1.00		+				WINNER	Colte	B.Sx3/8 bent	345	Н
11   12   12   12   12   12   12   12	Ĺ	Waterson	Wilking	Septed	36			W23x34	Cote	Lechild	14.5	1.00		25	×	*	æ	WHENTE	- Karte	8.5x3/8 bent	345	Н
Wilking Single			W33k130	Seated		*		W27x94	Crafe	16(3(3)8	145	100		.50		+	*	Wassiff	Colt	8.5x3/8.hent	382	Н
1	Ĺ	Name and	W33k330	Seated		,		W22x84	Cole	1603038	14.5	1.00	*			9	-	WRESTE	Keite	85c3/8 bent	345	Н
WHY NATE (MINER)         WHY NATE (MINER)<		D. WALLEY	WINISTO	Seated				W27x84	Ca5e	L4c3x39	316	1.00	18	28		*	æ	WSS/135	698	8.5x3/8 bent	2.5	
Marian   M	Ľ	200	WXXXXX	Searce	*		*	W22x94	4995	(40,3138	14.5	1.00		×		4	*	Washie	Shile	85438 bent	58	Н
WHATEM         WHATEM         Seated         1         Color         Color         2         0         Color         Color         2         0         Color		004004	WINIS	Seated			-	W27x84	Cale	16/3/38	34.5	100		7.	3	4	-	W38x135	Kale	85438 bent	345	Н
National States   S	8		W33×130	Jested				W23x94	Knite	14/3/38	14.5	1,00		×			٠	W266135	Kafe	& Soc3/8 bent	520	-
WHAND         WHAND         Samuel         Samuel         NEADON         NEADON <td></td> <td></td> <td>WINISH</td> <td>Search</td> <td></td> <td></td> <td></td> <td>962258</td> <td>Cake</td> <td>16009</td> <td>546</td> <td>1.00</td> <td></td> <td>5</td> <td></td> <td></td> <td>٠</td> <td>WHENTE</td> <td>finite.</td> <td>8.5/d/8.bem</td> <td>345</td> <td>Н</td>			WINISH	Search				962258	Cake	16009	546	1.00		5			٠	WHENTE	finite.	8.5/d/8.bem	345	Н
WY 1000         VARIANDO	L	OZD-SZO	WINISH	Seated		7	1	W23/94	Kistle.	Leonals	14.5	1.00	O.	×		×	e	WHENTE	Cuite	B.Exd/B.bent	345	Н
WHATEMED         WEATH DESTITION         State of the color         Color         London         44.8         10.0			WINISI	Sented			+	W22584	Code	(6003)	14.5	1.00		8		4	90	W266175	10496	B.Sod/Bleen	24.5	_
Windle   W	Ĺ	2007	W33c30	Seated		¥		W27x94	Smb	14/2/25	14.5	100				*		WINNELDS	Kelk	850/8 bent	ME	Н
WHATEMER         WHATEMER         CALCAL DESCRIPTION         HATEMER         TOTAL DESCRIPTION         TOTAL DESCRIPTION <td></td> <td></td> <td>Withtie</td> <td>Seated</td> <td></td> <td></td> <td></td> <td>W27i34</td> <td>Colle</td> <td>Lecksittle</td> <td>145</td> <td>1.00</td> <td>٠</td> <td></td> <td></td> <td></td> <td>*</td> <td>WHENTE</td> <td>Kulle</td> <td>85038 Seit</td> <td>57.0</td> <td>Н</td>			Withtie	Seated				W27i34	Colle	Lecksittle	145	1.00	٠				*	WHENTE	Kulle	85038 Seit	57.0	Н
Windley 1001         State           Lindball State           141.5         170.0         1         170.0         141.5         170.0         1         170.0	1	Widology	W33x130	System	7)	,	4	W27x54	Colt	16000	14.5	100				*		W36436	Kodh	B.Scolibbert	XX	-
WHANDAY         State of the common comm		S VOSCO	Withrittl	Seated				W22584	Colfe	(4chq)	311	100					-	WHISTE	Kash	85038 Sent	24.5	Н
	15	W148-230	WINTERS	Seated	æ	y	90	W22694	0.06	Lecht)(0)	34.5	1.00	w	9	000	*		W70x135	Kalk	8.5-0,78 been	345	Н
			WINTED	Seated	40			W27:94	Code	16008	316	1.00						WINNESS	Kalle	ASOJS best	345	Н
Mary 1201         Mary 1201         Amount 1201         <	WT4s	730 + 2 SDER	WINISHING	Seated	(2)	1	33	W22/94	Kelk	16/3/3/8	345	1.00	33	33	33	22	*	W36x150	Calle	ILSxijfi bent	345	Н
		3901	W33430	Teated	Ŧ	r	00	W27/04	Krite	(46/3/3))	14.5	100	X	*	*	30		W36x150	Kulte.	8.5c2/8 best	345	Н
	WYSK	730+25DER	W33k130	Seated	*	*	,	W27x94	Cole	160039	14.5	100		0		,		W396150	Knih	85/0/8 bent	345	4
		266	WIDING	Seated	7	Ţ		W27x94	Colte	140,018	346	100	4			,		W26x135	fah.	8.5x3/8.5em	345	-
	WT6x	_	WDkt00	States	,	Y		W27x54	Code	(400)	3.8	1.00		×				Willerth	Kele	MSc2/8 bent	52.	4
		2001	WORLD	Staffed	4	,		W27/94	Cale	1443638	14.5	100		,				Washis	Sale	85/0/8 bent	N.S.	4
	W14x	730 + 2 SDEN	WDetail	Jeathed		v.	5	W2564	Cole	88(x(xy)	316	100	5	9			5.	Wiletas	Koh.	(LSc2)@ bent	N.S.	+
			WINTER	Sealth	200			W27594	Keeth.	(4c2c3)	4.5	100	5		S			W No. 125	100	B NO. III Seen	343	+
	W14x	730 - 2 SDER	W33k130	Seated				W22/94	Cook.	Lectories	145	100		,			:	Wateras	Kelk	8.5x3/8 best	38	+
		+	WINTED	Seated	•			#2/294	No.	0,000,000	44	100	9	S	6	e	e.	W port 25.	100	School Seed	4	+
	W14s.	+	Water	September 1				M03/08	470	(60,000)	277	90.0						With the	100	0 50 50 50 50 50 50 50 50 50 50 50 50 50	27	+
		+	Market	Canad				WO Juda	473	16/3/36	275	000					-	Millorite	400	a Dodd Lean	376	+
		4	WINTER	Seated				W23-64	i de la	Lechilli	576	1.00						W36x135	Kest	R5x38 best	345	+
	WYAN	130 + 2 125 01	W33430	Search	*			W23394	Code	Lechall	34.5	1 00		*	*	•	-	WESSTOR	Cale	85x3/8 bent	24.5	⊬
		200	WINCES	Seated				WG7:94	Code	16/3/38	345	1.00						WHENTE	Kalk	&5x3/8 best	345	H
	Witer		W30c173	Chife	SERVE	23.5	195	W36x160	Kelle	Levert7	305	2.10	100	38	3	37	*	W28e346	Colt	14ch/39	115	Н
	00000	3843	W22x146	Cute	L4c3c3/8	17.5	121	W24c106	CHE	Leckijs	H.S.	1.00				30	*	W21430	Kelk	7x3/8 best 6.75x3/8 best	-12	Н
	757467	30+2 SDER's	MS-SA	Cole	Likelin 1/2-7/16 webi	215	308	W30x211	Cake	14 s4 s 12 - 2/16 sebil	215	2.08	,			0	2.	WZZK15Z	KNE	8238 641	23	Н
TABLE STATES OF THE TRANSPORT OF THE TRA		_	W39/230	Knife	1454x1/2-7/16 well	98	2.90	W30ct73	Krish.	14×4×1/2-1/16 weld	z	232	-	e i	*	1	4	W20x308	Colt	Br3/8 bent	22	4
	WTANZ	- 1	W30c245	Kn%	L5c0x1/2-7/16 well	17	2.00	W30c150	490	15ch 28	×	1.66	W12x19	Culte	L4x2x38	12	0.63	*				-
		2002				,			1	4	,		,	,								Н
	S =	ale factor l	isted for e	ach knife c	onnection correspon	ds to ratio	of depth t	o the 14.5"	deep conne	ction analyzed by Inc	raffea for a	axial capa	city and s	tiffness								

## INTERIOR COLUMN CONNECTION TYPE CATALOGUE - COLUMN 80

			North					thou.					East					West		
Column	Seine	- Trans	Š	Lemanth End	- Leave	Seam	fuse	5	Lemath Grid	Canal.	Seam	Type	5	Length End	Seem?	Even	fyte	5	Less the Less	Taxon.
Witedio	W3069	Cult	LArzellis	515	1.00	W24e63	Contraction	CACHERR	311	100						WOORS	Header PI	9x3/8	¥	
	W24d6	Code	L4x3x370	14.5	100	W24e62	Coste	(4c)(30)	14.5	100		62			108	W27584	Coh	7 \$43/\$ Sent	10.5	1.28
W14c39	W22694	Khite.	LikyJvJJB	S#	100	W24e52	fok	Lechtlis	576	100						W2764	Kale	7.5x3/8 bent	18.5	128
1000	W2264	Cole	1443438	3#	100	W24462	Cole	Lechilli	311	100						WZZS4	Kinik	7.5c3/8 best	185	128
V154x145	W22/64	Knife	14x3x3/3	345	100	W24462	Colfe	Lech300	342	100	œ	œ	3%	36	38	W27x54	Kalin	7.5c2/8.bent	18.5	128
- Marie Cast								4.											•	*
W14x133	W23-84	Ch.St.	14x3x3/8	316	1.00	W24e62	Knite	(6/3/33)	3116	1.00				÷		W27x54	Cult	7 Sodill bent	281	121
	÷	*	t	,	×	ж	1		÷	8	8	÷	*	Ŧ	*	1.	*		×	*
W14c233	W27-64	Code	L4x 3x 338	545	100	W2462	Code	(6/3/3)	345	1,00		,			,	W2344	Kaste	7.5x3/8 bent	18.5	128
100000000000000000000000000000000000000	W22/84	Kush	L4x3x3/8	346	100	W24662	En.St.	(4c)(10	345	100	98	æ	538			W2254	Key.	2 Sol/8 Sent	18.5	128
W14x283	W22694	Krift	24x3x30	388	100	W24s42	Code	(600)	345	1.00	1	3		,		W2354	Kale	7 Sodilli bent	- 581	128
	W2764	Code	CASESS	14.5	1001	W24e62	Cale	Leonge	14.5	1,00		,		,		W2754	SAME	7.50(8 bent	18.5	128
W14c322	W23/64	Chille	LACSEJB	346	100	W24662	Civile	(600)	316	1.00	8	ž			*	W23494	Kolk	7.5cl/8 ben	18.5	128
and the same	W23:64	Colt	L4x2x378	145	1.00	W24e62	Colt	16000	14.5	1.00			50			W23594	Kaile	7.5c3/8 bent	10.5	128
W146.258	902368	Knife	NOG/B	345	100	W24e52	Dafe	(tead)	14.5	100	2.5					W27/54	Kafe	75/3/8 bent	18.5	128
1000	W23/84	Chife	DACKER	145	100	W24eG2	Code	0(000)	14.5	100	.5	8		*	*	W27x54	- Kash	75c3/8 best	10.5	128
WINNER	W22d4	Code	LACROTTS	345	1,00	W24e62	Code	(6000)	14.5	1,00						WZN54	Colte	7.5x378 bent	18.5	128
	W27/84	Code	L4x3x3/B	345	100	Works	Colte	80000	316	100						W27954	Kulk	7.5xd/8 bent	18.5	128
000000	W22/94	State	DESIGN	325	100	Water	fok	00000	376	100						W2764	Kelk	7.5c3/8 bent	18.5	128
talan error	W22d4	Cult	14000	14.5	1.00	Without	fah	Lechalli	311	1,00	,	,			,	W2754	Kalk	7.5c3/8 bent	18.5	128
00000 A	W23/94	Knite	L4x3x3/B	34.5	1.00	W24662	Crite	Lechalls	316	100		×		•		W2754	569	7.5x3/8 bent	18.5	1.28
- Christian	1027-54	Code	14x3x378	14.5	100	W24e52	Code	16008	34.5	100				+		W27x54	faith.	7.5x3/8 bent	18.5	128
	W22584	Knik	24×3×3/8	14.5	100	W24e62	KNS.	\$4c3x335	345	100	12		122			W27x54	Colle	7.5x3/8 bent	18.5	122
Witeedall	W22/64	Krife	L4x3x3/8	345	100	W24402	Krafe	Lec2030	14.5	100	35	.8	- 00	(8)	(6)	W2759	Kalin.	7.5x3/lb bent	10.5	123
	W23-014	Corke	(4x2x378)	14.5	1.00	W24m52	- Knth	(4003)	14.5	100	,					W27494	Soite	7.5/2/3 bent	18.5	128
W14x730	W2254	Coste	Lecholyth	14.5	100	W24668	Code	L4c3v39	14.5	1,00		œ	36	(e	æ	W3043	Kaste	7.5x3/8 bent	18.5	128
	W2264	Cult	(4x3x378)	345	100	Wzaelik	Code	56000	34.5	100	F)	8	+11	ř.	*	Woods	Kalk.	7.5cd/8 bent	18.5	123
W14c70 + 2 WEB Pt	W27/84	Kn3r	Like he hills	386	100	WSteels	Cake	(4003)	348	1.00						W0069	Solt.	7 Sold best	286	1.28
17.001	W25/84	510	(4x3x3)3	14.5	1,00	W24662	CNS	Lechtiff	14.5	100				,		W27x54	Kase.	7.5x3/8 bent	18.5	128
W144730 + 2 WEB PI	W22/64	Cult	L4x3x3/8	14.5	100	W24e62	Cut	(40.53)	14.5	1.00	**	*	•	*		W27x54	Kalls	7.5c3/8 bent	10.5	128
12,592	W22s84	Colfe	54×2×1/8	545	100	W24462	Criste	(60/3)	34.5	100	ŀ	7		÷	7	W27x94	Calls	7.5x378 Sent	16.5	138
W14x330 + 2 WEB PR	W27/84	Knife	[4c2c3/8	34.5	100	W24x62	Koh	Lechilli	14.5	100					-	W27x54	Kak	7.5x3/8 bent	18.5	128
12.000	W27/64	Culte	CACOCID	14.5	100	W24m62	- Code	Lechalli	14.5	1,00						W23594	Colte	7.5x3/8 bent	18.5	17
W14c730 + 2 WEB Pt	W23x84	Crafe	C4c2c3/B	14.5	100	W24e62	Colt	(400/3))	14.5	100					2	. W27584	Kosh	7.5x3/8.5ext	10.5	1.28
17.546	W27cs4	Culte	LACACIDS	14.5	100	W24e62	Knk	16000	14.5	1.00				٠	,	W27x54	Cale	7.5x3/8 best	18.5	128
W14-720 + 2 WEB PI	_	Colfe	14×2×3/8	14.5	100	WS4eG3	Colt	(600)	14.5	100		,				W27x54	Knife	7.5x3/8.bent	16.5	128
12.565	W22/04	Code	(4c)c)(I)	365	100	W24662	Cafe	Lecion	14.5	100	æ	æ			2	W27/54	Kelk	7.5cd/lb bent	10.5	128
W144730 + 2 WEB PI		Code	(4c)c)()	14.5	100	W24e62	Code	16008	14.5	100		6				W25ste	Kuth	7.5c3/8.bent	18.5	128
12566	W27-84	Knife	CArSelffi	14.5	1.00	W24e62	Knife	(44343))	34.5	1.00	2	3		ï	÷	W27x54	Cult.	7.5x3/8 bent	14.5	128
W14x720+2WEBPI	W27-64	Krish	C4x3x3/B	145	100	W24e02	Knite	(60.00)	145	1,00	8	8			(4)	W27x54	System		2	2
12588	WZYOR	Colfe	Livracitis	511	100	W24m62	for	(600)	14.5	100						W27x54	Stated			1
W14x730 + 2 WEB PI	W36ct60	Cook	L4x4c3/8	3%	1.63	W3069	Contr	(444)))	215	1.62	M	y.	35	36	:5	150355.42	Kaile	2.5x3/8 bent	5.	135
12548	W24x104	Cult	(4x3x3))	345	100		1	+:	ti	*	P.	ř.	* 1	70	+	11055 #2	Sugle Angle	7 fod ill bent		*
W144730 + 2 WEB R	W30c211	Cn%	(442k1)2-7/16 weld	5	200	W30c123	Cole	L4s2x12-3716 weld	x	2.32						18055 #2	Truss Ons			1
17,200	Worth	M-Conn	,	,		W18c35	M-Conn		*	000	W20x211	Colt	SSx1/2 best 3/16 wedd	z.	232	W22554	100	65x1/2 Sent	£	22
W14x730+25DER	W39-150	Kult	L*xx3B	X.	365	Wastas	Knith	12003	12	1.00	W12:19	Ente	(44243)		250				95	5)
2007																				

No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity
Where no beams size is indicated no member is framing into the column
The Information specified for each connection was taken from the latest corresponding frankel Steel Limited steel shop drawing for that connection

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OLUMN 81
ATALOGUE - C
TION TYPE C
UMN CONNECT
INTERIOR COLI

	Witterloom   Steps   Type	(1989) (1985) (1	60.37 (0.07) (0.	973-944 W23-944 W23-944 W23-944 W23-944	Type	ž					Length [m]	Factor?	Esam	fype	2	F-10 Mar. 1	
1	WHEND   WORNER   COAP	185 185 185 185 185 185 185 185 185 185	0.73 0.73 0.74 0.75	2002-04-4 2002-0	50.0	600					4				200	[Millian]	Factor?
1	Weekly   Weekly   Cock	1115 1115 1116 1116 1117 1117 1117 1117	0.73 0.73	#25/20/ #25/20/ #25/20/ #25/20/ #25/20/							11.5		W24e68	Heater	(44295216	415	
1	With table	1115. 1116. 1117.	0.75 0.75	W2764 W2764 W2764 W2764 W2764							11.5	128	W24x84	Header	Laxbif/16 best-Uschif/16 best	14.5	3
1	W. No. 1964   W. No. 2004   Co. 10.	115.5 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 6.73 7. 10 7. 10	W27/84 W27/84 W27/84 W27/84		- 4	4				345	- 0	W33x130	Header	14c3/38 Seet	g	•
No. 11   N	With title	115. 115. 115. 115. 115. 115. 115. 115.	0.74 0.74	W2764 W2764 W2764 W2764	Seated	9					115		W27x84	Header	(4x3x5/16 best-15x3x5/16 best	145	*
1	Website	11.5 11.5 11.5 11.5 11.5 11.5 11.5 11.5	0.03 0.03	W27484 W27484 W27484	Seated	36	***				11.5	æ	W27x84	Header	(4x3x5/16 bert-L5x3x5/16 bert	342	*
1.0.   1.0.	Wield   Wiel	165 165 165 165 165 165 165 165 165 165	079 079 079 079 079 079 079 079 079 079	W2764 W2764 W2764		•	-	+	-	+		*			4.0	* 2	:
1	W   W   W   W   W   W   W   W   W   W	11.5 11.5 11.5 11.5 11.5 11.5 11.5 11.5	0.79 0.79 0.79 0.79 0.79 0.79 0.79 0.79	W27484 W27484 W27484	Seated	2	-			-	115		W23464	Header	Na information Available	92	ż
Part	W.   W.   W.   W.   W.   W.   W.   W.	115 115 115 115 115 115 115 115 115 115	0.73 0.79 0.79 0.79 0.79 0.79 0.79 0.79 0.79	W27484 W27484	N	36	*	+	+	+	+	*				90	*:
Part	W.	11 15 16 16 16 16 16 16 16 16 16 16 16 16 16	0.09 1.00 1.00 1.00 0.03 0.03 0.03 0.03 1.00 1.00	W27x84 W27x84	Seared	PS	b	+	+	+	115	,	WZZNBA	Healor	No inferentian Available		
WHATE                 CARRELLY CARR	Wisheld   Wisheld   Cock	6 6 7 7 7 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8	0.01 1.00 1.00 0.07 0.07 0.07 1.00 1.00	W27x84	Stated	4	it.			-	115.	3.	W23x84	Header	No Information Available	4	2.
	WY6471   WY8452   GOAR	7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7	001 001 400 400 400 001 001 001	-	Seated		ti	1		-	115		W27s84	Header	No Information Available		
Part	Withold   Without   Cock	2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	001 079 079 079 079 100 100 100	W27x84	Seated		4				115	7.	W27x84	Heador	L5-G5-G7-16 bent-L4x23-G7-6 bent	511	
Part	WY4,019   WT04442   Ciche	2 I I I I I I I I I I I I I I I I I I I	0.79 0.79 0.79 0.01 0.01 0.01	W27484	Systed		9				115		W23464	Header	LS/04/2/5 bert-14/04/3/16 best	145	
Part	Wheels   Worked   Cock	2	0.79 0.79 0.09 0.00 0.00 0.00 0.00 0.00	W2744	Seated		*);	+	-		11.5		W27x84	Healer	LExibility best-Like In 5/16 best	14.5	
Part	VY   MACK   VY   VY   VY   VY   VY   VY   VY   V	2	0.79 0.79 1.00 1.00 1.00 1.00 1.00	W27s84	Seated	×	*				115		W27x84	Heafer	LEXING/16 best-L4r2nS/16 best	14.5	3
Fig. 15   Fig.	With teach   With teach   Contect	2	100 100 100 100 100 100 100 100 100 100	W27x84	Seated	30	*				115	180	W27x84	Heady	LSxDsf/16 bert: L4x2k5/16 bert	14.5	
Printing                 Control                  Control                      Control                     Control                     Control                     Control                     Control                     Control                     Control                     Control                     Control                     Control                     Control	W14-645   W24-647   Gick	2 2 2 2 2 2 2 2	8 8 8 8 8 8 8	W27s84	Seated		2.		-		115		WZNBA	Healer	LSx2xS/16 best-L4x2xS/16 best	345	
GRAND         CARD         CARD <t< td=""><td>  100 cm   1</td><td>X X X X X X X X X X X X X X X X X X X</td><td>100</td><td>W27x84</td><td>Seated</td><td></td><td>*</td><td></td><td></td><td></td><td>115</td><td>*</td><td>W27s84</td><td>Heador</td><td>(5i3i5/16 best-(4i3i5/16 best</td><td>14.5</td><td>÷</td></t<>	100 cm   1	X X X X X X X X X X X X X X X X X X X	100	W27x84	Seated		*				115	*	W27s84	Heador	(5i3i5/16 best-(4i3i5/16 best	14.5	÷
(4) (4) (4) (4) (4) (4) (4) (4) (4) (4)	WY 64-00   WY 60-645   GAN	25 25 25 25 25 25 25 25 25 25 25 25 25 2	100	W27494	Seated						115		W27x84	Heater	US/03/5/16 best-bic/3/5/16 best	14.5	
Wished         Good         Display         Single         Noted         Internal (1000)	Withold   Without   Coche	11 11 11 11 11 11 11 11 11 11 11 11 11	100	W27x04	Seated	9	9				115	,	W27x84	Header	15x3x5/16 bent-Like3x5/16 bent	14.5	
No.	WY 14-00   WY 14-00   Cock	2 4 1		W27x84	Stated	٠	9(				115		W27x84	Header	L5ch5/16 best-(4c3r5/16 best	14.5	÷
WHAND         WHAND         Chicago         Liberal of the control of the co	Wileyed   Wileyed   Gook	11 11	620	W27-84	Seated		*	1	-		115		W2758a	Healer	(5x3x5/36 best-14x3x5/16 best	345	•
WHOME         Cond.         Cond. <th< td=""><td>  Vivience   Vivience</td><td>11.5</td><td>629</td><td>W27x84</td><td>Seated</td><td>Œ.</td><td>Œ</td><td>+</td><td>-</td><td></td><td>115</td><td></td><td>WZZHB4</td><td>Header</td><td>LExth5/16 benc.L4ch5/16 bent</td><td>5#</td><td>i.e</td></th<>	Vivience	11.5	629	W27x84	Seated	Œ.	Œ	+	-		115		WZZHB4	Header	LExth5/16 benc.L4ch5/16 bent	5#	i.e
Winding   Cock	N14400   N74400   CONN		670	W27484	Seated	20	E	+	-		11.5	8	W27x84	Header	L5x3xf/16 best-14x3x5/16 best	34.5	ž.
Weight                 Total and single control of the c	Viveoria   Viveoria   Coche	11.5	626	W27594	Seated	E	E	+	+	1	11.5		W27584	Heater	Uschilf 16 best-lechtlift best	24.5	,
Winds                 Color                 Liberation                 18.5                  19.5                  19.5                  19.5                  19.5                  19.5                  19.5                  19.5                  19.5                  19.5 </td <td>  W. N. S. S.</td> <td>145</td> <td>100</td> <td>W27x94</td> <td>Seated</td> <td>Ç6</td> <td>):  }</td> <td></td> <td></td> <td></td> <td>11.5</td> <td>*</td> <td>W27x84</td> <td>Header</td> <td>L5x2x5/16 best-14x2x5/16 best</td> <td>14.5</td> <td>÷</td>	W. N. S.	145	100	W27x94	Seated	Ç6	):  }				11.5	*	W27x84	Header	L5x2x5/16 best-14x2x5/16 best	14.5	÷
Winking                 Original                 Clash Blanck-Rachal Strategy                 Teach Strategy                  Teach Strategy                      Teach Strategy                       Teach Strategy                       Teach Strategy                       Teach Strategy                       Teach Strategy                       Teach Strategy                       Teach Strategy                      Teach Strategy                        Teach Strategy                       Teach Strategy                       Teach Strategy	W146070   W274641 Cash   W14607   W24641 Cash   W24641 Cash   W24641 Cash   W24641   W24641   Cash   W24641	34.5	100	W27:54	Seated	*1			-	+	11.5	*	W27x84	Healer	LSiGnE/16 Seet-L4r2hS/16 Sent	14.5	<u>*</u>
Winding   Winding   Control   Cont	WY46/30 WY464/3 Code WY46/30 WY464/3 Code WY46/30 WY46/3 Code WY46/30 WY46/3 Code	27.75	100	W27584	Seated	*	,	+	+	+	11.5		W27484	Header	US42x6/16 bent-Like2x6/16 bent	345	
	WY84770 W24462 Kishe WY84770 W24462 Kishe W24462 Kishe	6 77	100	WENNE	Served	+	+	+	+	+	511		W.C.Wes	Heater	LEADING TO DESCRIPTION OF THE PARTY OF THE P	14.5	
	W146770 W2465 Knits W2465 Knits	2 2		Marine	- Control		+	+	+	+	911		MOTOR.	Heate.	LEADING IN ORDER AND THE PARTY IN COURT	277	
	W16x720 W24xt2 Smith	775	000	Wohele	Control	9	2	t	+	+	116		Wohen	Header	Harbert Short Sarbert Short	375	
Part		14.5	100	W27sh	Seated			H	+	+	118		WZNS	Header	LSGM216 bend-lat/2016 bent	345	
15th	Wester Table 2 Wildlife WZ And Z Confe	11.5	0.79	WZNSA	Sealed				H	-	11.5	2	W27584	Healer	15x3x5/16 best-(4x3x5/16 best	316	
Wind   Supple   Sup	12.5x1 W24452 Guife	111.5	67.0	W27e84	Seated	*		H			11.5		W27x8A	Header	USON/TH best (4cht/16bert	145	×
No.	WHACTO+2 WERR WINNESS SUM	511	629	W27x84	Seated	9	9				115		W27x84	Heador	L5x3x5/16 bent-L4x3x5/16 bent	14.5	
Windle   W	12.5x1 W24x02 Kally	115	67.0	W27x84	System	26	36				11.5	25	W27x84	Heafer	Updhf/16 bere-L4c2h5/16 bert	345	×
No.	WYAR739 + 2 WEB PI WRANG? Knith	11.5	640	W27n84	Seated		2				11.5	0.79	W27x84	Knft	14c2c39	14.5	100
Windows   Color   Co	12.5x2 Wydeli2 Knife	118	620	W27x84	Seated		Œ	+			115	0.29	W23484	Colt	teoon	316	1.00
	WY4c730 + 2 WEB FI W24x62 Kish	115	640	W27x84	Seated	(8)	*				311	0.78	W27x64	Koh	(4c)c)()	345	100
	17300 MORRED Crisis	118	678	WZPHR	Seated		+	+	+		115	0.79	Wahile	for	Mench	14.5	100
	WHALTO 4.2 WEB PIT WITHOUT KNOWS	22.5	162	MS-26	Double Fin	H15638	+	+	+			3	WZZette	Coh	344439		146
	+ + court	+	+	105-61	Double Fin	F1 8×5/10	+	+	ť		8	103	W32x130	Colt	(4+4<)0	12	146
WHATEN 2-20EEPT   WHATEN 2-2	W14c730 + 2 WEB Pts W30c173 Knth	+	222	25-696	Daylor hm	24 8 KERS		+	+				WESKANI	Double Frit	H+273cfgs	27.5	:
With the connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraftea for axial capacity and stiffness  No factor listed for each knife connection types as they are conservatively assumed to have sufficient tensile capacity.	Mary Mary			Message and	400	14-5-10	+	+	+			400					
1 Scale factor isted for each knife connection corresponds to ratio of depth to the 14.5" deep connection analyzed by Ingraffea for axial capacity and stiffness  No factor listed for other connection types as they are conservatively assumed to have sufficient tensile capacity	W14770 + 2 WEB PEL Manual and Manual Annual	1		2000		2010000		t	1		2 2	0.00				999	
	ı			,	,			NA N	CANO.		44	2,12	,	,	*	+	
NO TACKOT ISSECTION OFFICE CONTRIBUTION TYPES AS THEY ARE CONSERVATIVEN TO THE STATE OF THE STAT		esponds to ratio	of depth to	the 14.5"	deep connec	tion analyzed by In	yraffea for axia.	capacity a	and stiffness								
	No factor listed for other connection types as they are	ey are conservativ	rely assume	ed to have	sufficient ter	isile capacity											

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**Guy Nordenson and Associates** 

C6.0 INTERIOR COLUMN BRACING CAPACITY TABLES

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

MN 58 (NORTH – SOUTH)	
ACITY CATALOGUE - COLUI	
JR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 58 (NORTI	
C6.1 INTERIO	

Hoor As built Axial	Design Axial	Design Axial	Reg Dohn Axial		As-Design	As Designed Axial Capacity of Bracing Connections (Kips)	Reacing Connection	ons [Kips]			(Desirated by No. Consistence Dalid	Descripted by the Consentation Dalid	
	, T	(Increments)	Wed	Obese Tales	North Connection	of Issue	Obes fame	South Connection	C Boot	North in Tension +	South in Tension +	Minimum (kip)	Percentage of Req.
ldNI Port	Idea)	della	IMPI	COHIL TYPE	1,000	1430	West.	(divi)	- Inpl	South in Comp. In pa	reorm in comp. (appl	900	Dracing Capacity
Floor 42 134	333	232	4.4	Name of	0	0	Knife	- 50	26.0	188	30	i do	1263%
ŀ	455	201	1.6	Header	73.3	326.1	Knife	1.8	84.0	151	320	151	1730%
Hoor 45 637	579	124	11.6	Knife	12.4	126.0	Knife	8.1	84.0	36	134	*	83298
Floor 44	701	123	14.0	Knife	12.4	126.0	Knife	1.8	84.0	*	134	*	848/WE
Floor 43 989	825	124	16.5	Knife	12.4	129.0	Kulfe	8.1	84.0	.96	134	98	SBAN
Fluor 62	947	122	16.9	Knife	124	1260	Knife	8.1	84.0	96	134	96	60908
Floor 41 1141	1/01	124	21.4	Knife	12.4	1260	Knife	8.1	0.88	96	134	乐	450%
Floor 40	1194	123	23.9	Knife	124	126.0	Knife	-50	84.0	38	134	98	40406
Floor 39 1393	1270	77.	25.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	37906
Floor 38	1393	123	27.9	Knfe	12.4	126.0	Knife	8.1	84.0	96	134	96	346%
Floor 37 1845	1518	124	30.4	Knife	12.4	1260	Kinte	8.1	84.0	96	194	. 96	316%
	1641	121	37.8	Knife	12.4	1260	Knife.	121	84.0	98.	134	筆	294%
Fluor 35 1897	1769	125	35.3	Knife	12.4	126.0	Knife	8.1	84.0	98	134	98	273%
Floor 34	1809	124	37.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	255%
Floor 13 2149	2014	125	40.3	Kaife	12.4	126.0	Knife	8.1	94.0	96	134	*	239%
	2139	124	42.8	Knife	12.4	125.0	Knife	8.1	84.0	36	134	*	225%
Fleept 31 Z401	2264	126	45.3	Kaife	12.4	126.0	Knife	8.1	84.0	96	134	96	21396
Floot 30	2309	128	478	Knife	17.4	125.0	Knife	8.1	84.0	989	134	*	202%
Hoor 29 2653	2516	128	603	Knife	17.4	178.0	Knife	1.80	840	#	13.6	38	335%
	2641	125	52.8	Knife	12.4	1260	Knife	8.1	84.0	98	134	35	182%
Fleor 27 3905.	2768	121	55.4	Knite	12.4	1260	Knite	8.1	84.0	36	134	98	174%
	2894	126	679	Knife	12.4	126.0	Knife	8.1	84.0	96	134	88	167%
Floor 25 3157	3021	127	60.4	Knite	12.4	1260	Knife	8.1	84.0	96	134	審	100%
	3147	176	679	Xnife	124	126.0	Knife	8.1	84.0	36	134	£	15,9%
Fluor 23 3409	3307	160	66.1	Knife	124	126.0	Knife	8.1	84.0	98	134	19	146%
	3466	159	169	Knife	17.4	1750	Knife	8.1	84.0	3	138	36	139%
Floor 21 3723	3627	161	322	Knife	12.4	1260	Knife	81	84,0	385	134	Œ.	133%
+	3758	131	75.2	Knife	12.4	1250	Kuife	8.1	84.0	96	134	8	1289h
Floor 19 4006	3091	134	77.8	Knife	12.4	1260	Knife	8.1	34,0	38	134	98	124%
	4019	128	80.4	Knife	12.6	1260	Knite	8.1	84.0	96	134	190	120%
Floor 17 4258	4148	129	83.0	Knife	12.4	1280	Knife	8.1	84.0	98	134	*	11696
	4277	129	88.5	Kaite	124	126.0	Knife	8.1	84.0	38	134	96	113%
Hoor 15 4510	4407	130	88.1	Kaife	12.4	126,0	Knife	8.1	84.0	.96	134	98	10908
Floor 14	4535	129	90.7	Knife	12.4	1560	Knife	8.1	94.0	96	134	98	100%
Floor 13: 4752	4666	130	93.3	Knife	12.4	126.0	Kuife	8.1	84.0	36	134	K	103%
	4795	130	95.9	Knite	12.4	1260	Knife	8.1	94.0	96	134	96	1000%
Rior 11 5014	4927	131	911.5	Knife	12.4	1260	Knife	8.1	84.0	36	134	98	9666
	1905	175	1012	Knife	12.4	1260	Knife	8.1	84,0	186	134	乐	8686
Roor 9 5266	5165	131	103.8	Knife	12.4	126.0	Knife	81	0.40	386	134	98	93,54
	5323	130	106.5.	Knife	124	126.0	Knife	8.1	84.0	98	134	96	946
Hoer 7 5518	1	6	4		6	· ·	C.	,	i	Y.	ę.	*	, i.
Hoor 6	,			,	,		X	) I				1	X
Hoor 5	1	1		4	ť	X		ť		*	Ý	1	4
Roce 4		0		1.0	9	(	,	9				(	
Boor 3								- 1		-			
These h		1				7							

Floor As	As built Axial Column brads.	Design Axial Column Loarls	Design Axral Column Lrad	Reg Conn Axial Capacity (2%)		As Desig	ned Axial Capacity o	As-Designed Axial Capardty of Bracing Connections [Kips]	ns (Mps)			Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)	cing Capacity in the North-South Director Provided by the Connections Gnly)	
	(Cumulative)	(Cumulative)	Uncrements)	Perel.	One for	West Connection	r lead	China fate	East Connection 7 feed	r (biol	West in Tension + East	West in Tension + East East in Tension + West	Minimum (Rip)	Percentage of Reg.
Rost	Mal	06	06	180	Header	16.6	172.5	Header	20.7	146.8	181	143 143	143	744.5%
Flast 47	134:	372	232	6.4	Header P	188	7007	Header	75.0	167.0	152	. WC	152	SHAME
Floor 46		455	ECI.	3.6	Header Pl	20.5	1.68	Header	283	£,631	210	WC	210	230746
Hoor 45	637	579	124	11.6	Header	12.2	1002	Header	362	2002	221	136	136	1170%
Floor 44		701	17.3	14.0	Header	12.2	1002	Header	16.4	122.5	135	117	117	832%
Elsor 43	686	825	124	165	Header	12.2	100.2	Header	36.2	7007	721	1361	136	827%
Elsor 42		947	122	169	Header	12.2	100.2	Header	16.4	1225	135	111	1117	616%
Fixor 41	1141	1/01	124	21.4	Header	19.2	100.2	Header	36.2	2008.8	221	1361	136	637%
Floor 40		1194	123	23.9	Header	122	1002	Header	16.4	122.5	135	117	017	4899
Eluor 39	1393	1270	37.	25.4	Header	12.2	100.7	Header	362	2001.8	331	136	136	537W.
Floor 38		1393	123	27.9	Header	12.2	100.2	Header	16.4	1225	115	117	117	41996
Floor 37	1645	1518	124	30.4	Header	12.2	1002	Header	382	2088	221	196	136	450%
Floor 36		1641	123	87.8	Header	123	100.2	Heater	164	172.5	135	1117	.117	356%
Fluor 35	1897	1765	125.	35.3	Header	12.7	1002	Header	362	WBB	121	136	136	389%
Floor 34		6881	124	378	Header	19.2	1002	Header	16.4	122.5	135	117	117	30506
Floor 33	2149	2014	125	40.3	Header	12.2	100.2	Header	36.2	2009	221	136	136	339%
Floor 32		2139	124	42.8	Header	12.2	100.2	Header	164	122.5	135	117	317	273%
Hept 31	2401	2264	126	45.3	Header	12.2	106.2	Header	362	2000	122	1361	136	30106
Floor 30		5309	175	478	Header	17.7	1002	Header	16.4	172.5	135	117	117	744%.
Hoor 29	2653	2516	178	503	Header	49.2	100.2	Header	362	2002	721	1381	136	271%
flass 28		2641	125	52.8	Header	12.2	1002	Header	16.4	5723	135	113	117	223%
Floor 27	3805	2768	121	554	Header	12.2	100.2	Header	36.2	2003	221	136	136	746%
Hoor 26		2604	126	57.9	Header	19.2	106.2	Header	16.4	122.5	135	117.	117	20296
floor 25	3157	1200	127	60.4	Header	12.2	1002	Header	36.2	2002	725	136	136	226%
Fluor 24		3147	126	679	Header	122	1007	Header	16.4	122.5	115	717	117	1898
Fluor 23	3409	3307	160	66.1	Header	12.2	1002	Header	36.7	2003	221	106	136.	200%
Floor 72		2466	159	69.3	Header	10.2	100.7	Header	16.4	1775	135	117	117	168%
Floor 21	3723	3627	161	27.5	lleader	19.2	100.2	Header	362	2001.0	- 221	106	136	186%
Floor 20		3758	131	752	Header	12.2	1002	Header	16.4	122.5	135	117	117	155%
Hoor 19	4006	3891	134	77.8	Header	12.2	106.2	Header	362	2002	721	132	136	175/6
Hoor 18		6009	128	897.4	Header	12.2	100.2	Header	16.4	1725	335	1117	117	3458
Floor 17	4258	4148	129	83.0	Nexter.	122	1092	Header	362	208.8	221	136	136	164%
Fluor 16		4277	129	85.5	Header	12.7	1007	Header	16.4	1225	135	117	117	136w.
Hoor 15	4510	4407	130	88.1	Header	12.2	100.2	Header	38.7	2088	221	136	136	15508
Floor 14		4535	128	90.7	Header	12.2	1002	Header	16.4	122.5	115	117	117	129%
floor 13	4762	4666	130	93.3	Beater	12.2	1002	Header	36.2	2093	-221	196	136	140%
Heyr 12	-	4795	130	959	Header	12.2	100.2	Header	16.4	122.5	135	1117	1117	1229
Bear 11	5014	4927	131	911.5	Header	12.7	100.2	Header	367	2003	37.1	136	136	f1996
Floor 10		1905	175	1012	Header	19.2	1002	Header	16.4	177.5	105	117	117	11500
Floor 9	5266	5162	131	103.6	Header	12.2	1007	Header	362	2007	221	136	136	13106
Floor B		5323	130	106.5	Header	122	1002	Header	164	122.5	136	7117	.00	T1098
Hoor 7	5518			4		,		· ·	Y	-	Y		3	
Hoore		ý	X		ý	,		ý	)m(	X			,	X
Hoor 5		i.	¥	+	,	Y	1	y	Y	4	x.	×	Y	1
Hoor 4			9			9	(	,						
Boor 3						É					+		+	
Hoor 2		1	Xi.				*	-	×	į,			X.	

Notes 1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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н – SOUTH)
RIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 59 (NORTH –
Y CATALOGUE - CI
SRACING CAPACITY CATALOGUE
INTERIOR COLUMN F
C6.3

Chamdalora  (Chamdalora  Chamdalora  Cha	Foor As built Astal	100	Design Axial Column Loads	Design Axmi	Reg Conn Axial Caracity (2%)		As-Design	ned Axial Capacity o	As-Designed Axial Capacity of Bracing Connections [Kips]	ns [Kips]			Total Bracing Capacity in the North-South Direction (Powaded by the Connections Only)	he North-South Director	N.
1949   1944   1949   1944   1949   1944   1949   1944	İ		Jamulative)	(Increments)			North Connection			South Connection		North in Tension +	South in Tension +	Affairment (bin)	Percentage of Reg.
13         18<		lds	[Kip]	IKipl	(Krip)	Conn Type	Thip	C Bool	Conn Type	Thip	Chipl	South in Comp. [ktp]	North in Comp. [kip]	Ideal manufacture	Bracing Capacity
266         577         6,17         Kidne         8,81         980.0         Kidne         8,91           417         378         573         6,17         Kidne         8,13         980.0         Kidne         8,00           418         378         578         6,15         Kidne         8,11         9,10         6,10           556         617         626         613         614         8,10         6,10         6,10           556         617         626         8,13         8,10         Kidne         8,0           667         624         526         613         Kidne         8,1         9,10         Kidne         8,0           667         624         526         613         Kidne         8,1         9,0         Kidne         8,0           667         624         527         10,2         Kidne         8,1         9,0         Kidne         8,0           868         744         547         547         10,2         Kidne         8,1         9,0         Kidne         8,0           868         744         547         10,2         Kidne         8,1         9,0         Kidne         8,0	-		140	5 5	200	Voite	0.0	640	Vole	00	0.00	31	10	100	27,000
201         590         542         Kode         8.1         8840         Code         6.0           470         379         592         542         Kode         8.1         8840         Code         6.0           480         471         593         644         566         6.0         6.0         6.0           553         644         596         645         Kode         8.1         840         Kode         6.0           657         644         596         14.3         Kode         8.1         840         Kode         6.0           659         644         597         14.3         Kode         8.1         840         Kode         6.0           659         644         597         14.3         Kode         8.1         840         Kode         6.0           659         744         548         Kode         8.1         840         Kode         6.0           878         747         14.2         Kode         8.1         840         Kode         6.0           1400         878         74         14.2         Kode         8.1         840         Kode         6.0           14	-		205	57	4.1	Knife	8.1	840	Kaife	09	630	31	3	17	173196
470         318         565         8,64         Kolde         8.1         84.0         Kolde         6.0           451         495         75         Kolde         8.1         64.0         Kolde         6.0           555         647         55         7.2         Kolde         8.1         64.0         Kolde         6.0           667         654         55         1.3         Kolde         8.1         80.0         Kolde         6.0           667         654         55         1.3         Kolde         8.1         80.0         Kolde         6.0           868         73         1.4         5.7         1.43         Kolde         8.1         80.0         Kolde         6.0           868         7.1         1.4         Kolde         8.1         80.0         Kolde         8.0         Kol		.00	362	15	52	Knife	8.1	84.0	Knife	6.0	610	11	96	1//	135,796
440         315         546         715         Mode         81         840         Kolde         60           556         481         566         487         566         66         60         60           650         554         56         487         Kolde         81         840         Kolde         60           650         554         56         487         Kolde         81         840         Kolde         60           660         571         54         56         487         Kolde         81         840         Kolde         60           660         774         57         1120         Kolde         81         840         Kolde         60           660         774         57         113         Kolde         81         840         Kolde         60           860         77         114         57         114         Kolde         81         840         Kolde         60           1100         804         74         114         Kolde         81         840         Kolde         60           1100         804         74         840         Kolde         81 <t< td=""><td></td><td></td><td>318</td><td>:95</td><td>6.4</td><td>Knife</td><td>1.8</td><td>84.0</td><td>Knife</td><td>6.0</td><td>63.0</td><td>7.1</td><td>96</td><td>717</td><td>1117%</td></t<>			318	:95	6.4	Knife	1.8	84.0	Knife	6.0	63.0	7.1	96	717	1117%
\$456         \$41         \$66         \$66         Reduce         \$81         \$810         Koline         \$60           \$567         \$548         \$56         \$103         Koline         \$81         \$810         Koline         \$61           \$670         \$671         \$54         \$16         \$103         Koline         \$81         \$810         Koline         \$61           \$670         \$671         \$17         \$14         \$16         \$61         \$81         \$80         Koline         \$61           \$670         \$171         \$17         \$14         \$66         \$81         \$80         Koline         \$60           \$671         \$17         \$14         \$66         \$81         \$80         \$66         \$60           \$140         \$17         \$14         \$66         \$81         \$80         \$66         \$60           \$140         \$17         \$14         \$66         \$81         \$80         \$66         \$60           \$140         \$17         \$14         \$66         \$81         \$80         \$66         \$60           \$140         \$16         \$10         \$16         \$10         \$66         \$10         \$60		.02	375	95	7.5	Knife	8.1	84.0	Kolfe	6,0	63.0	71	96	11	940%
8565         4817         Stofflee         81         9810         Kunflee         80         9810         Kunflee         80         9810         Kunflee         80         Kunflee         80         90         Munflee         80         90         Munflee         80         90         Munflee         80         80         80         80         80         80         80         80         80         80         80         80         80         80         80         80         80	25		431	95	8.6	Knife	8.1	84.0	Knife	- 60	63.0	7.1	96	7.1	626%
890         654         556         1105         Kidnle         8.1         84.0         Kidnle         6.0           802         701         57         13.1         Kidnle         8.1         84.0         Kidnle         6.0           803         714         57         14.4         Kidnle         8.1         84.0         Kidnle         6.0           804         627         14.4         Kidnle         8.1         84.0         Kidnle         6.0           804         627         14.4         Kidnle         8.1         84.0         Kidnle         6.0           1100         934         57         14.1         Kidnle         8.1         84.0         Kidnle         6.0           1100         947         5.7         14.1         Kidnle         8.1         84.0         Kidnle         6.0           1100         940         5.7         24.1         Kidnle         8.1         84.0         Kidnle         8.0           1100         5.7         24.1         Kidnle         8.1         84.0         Kidnle         8.0           1100         5.7         24.1         Kidnle         8.1         84.0         Kidnle <td></td> <td>5</td> <td>487</td> <td>95</td> <td>1.6</td> <td>Knife</td> <td>8.1</td> <td>84.0</td> <td>Knife</td> <td>8.0</td> <td>019</td> <td>7.</td> <td>96</td> <td>77</td> <td>730%</td>		5	487	95	1.6	Knife	8.1	84.0	Knife	8.0	019	7.	96	77	730%
869         501         557         13.20         Reline         8.1         84.0         Kunfre         8.0           879         774         54         14.3         Konfre         8.1         94.0         Kunfre         8.0           879         774         15.4         6.0         8.1         94.0         Kunfre         8.0           864         878         9.7         15.4         6.0         8.1         94.0         Kunfre         8.0           100         9.97         15.4         Konfre         8.1         94.0         Kunfre         8.0           1100         9.96         5.7         15.4         Konfre         8.1         96.0         Kunfre         8.0           110         9.97         15.4         Konfre         8.1         96.0         Kunfre         8.0           110         9.97         15.4         Konfre         8.1         96.0         Kunfre         8.0           110         9.97         15.4         Konfre         8.1         96.0         Kunfre         8.0         Kunfre         8.0         Kunfre         8.0         Kunfre         8.0         Kunfre         8.0         Kunfre <td< td=""><td>200</td><td></td><td>544</td><td>98</td><td>10.9</td><td>Kolle</td><td>8.1</td><td>84.0</td><td>Knife</td><td>60</td><td>0.68</td><td>21</td><td>360</td><td>216</td><td>#SANN</td></td<>	200		544	98	10.9	Kolle	8.1	84.0	Knife	60	0.68	21	360	216	#SANN
869         584         585         13.1         Golde         8.1         840         Kolle         8.0           860         774         57         143         Golde         8.1         5800         Kolle         8.0           864         627         114         Golde         8.1         5800         Kolle         8.0           1100         947         57         163         Kolle         8.1         680         Kolle         6.0           1100         947         57         163         Kolle         8.1         680         Kolle         6.0           1206         1705         57         171         Kolle         8.1         880         Kolle         6.0           1207         1717         58         27.3         Kolle         8.1         840         Kolle         6.0           1207         1717         58         27.4         Kolle         8.1         840         Kolle         6.0           1208         171         50         27.4         Kolle         8.1         840         Kolle         6.0           1208         27.4         Kolle         8.1         840         Kolle		32	109	15	12.0	Knife	8.1	84.6	Knife	0.9	929	71	96	71	592%
878         774         9.7         14.3         Korle         8.1         84.0         Korle         8.0           1964         828         9.7         11.64         Korle         8.1         84.0         Korle         8.0           1964         828         9.7         11.64         Korle         8.1         84.0         Korle         8.0           1190         9840         9.7         11.0         8.0         Korle         6.0           1190         9840         9.7         10.0         Korle         8.1         84.0         Korle         6.0           1190         9840         9.7         10.1         Korle         8.1         94.0         Korle         6.0           1190         9840         9.7         10.1         Korle         8.1         94.0         Korle         6.0           1191         9.1         10.0         Korle         8.1         94.0         Korle         6.0           1191         9.1         10.0         Korle         8.1         94.0         Korle         6.0           1192         11.1         5.9         27.1         Korle         8.1         94.0         Korle	101		199	.99	13.1	Knite	8.1	84.0	Knife	6,0	63.0	1//	96	1/	54196
17.1   5/3   14.5 4   Kinite   8.1   84.0   Kinite   8.1   84.0   Kinite   8.0     19.0   957   71.2   Kinite   8.1   84.0   Kinite   8.0     19.0   957   71.2   Kinite   8.1   84.0   Kinite   8.0     19.0   957   71.2   Kinite   8.1   84.0   Kinite   8.0     17.3   11.7   11.7   52.0   72.1   Kinite   8.1   84.0   Kinite   8.0     17.3   11.7   52.1   Kinite   8.1   84.0   Kinite   8.0     17.3   11.7   52.1   Kinite   8.1   84.0   Kinite   8.0     17.3   11.7   52.1   Kinite   8.1   84.0   Kinite   8.0     17.3   11.3   52.2   Kinite   8.1   84.0   Kinite   8.0     17.3   11.3   52.2   Kinite   8.1   84.0   Kinite   8.0     17.3   17.3   52.2   Kinite   8.1   84.0   Kinite   8.0     17.4   11.4   52.0   22.1   Kinite   8.1   84.0   Kinite   8.0     17.4   11.4   52.0   22.1   Kinite   8.1   84.0   Kinite   8.0     17.4   14.5   52.0   22.1   Kinite   8.1   84.0   Kinite   8.0     17.4   14.5   52.0   22.1   Kinite   8.1   84.0   Kinite   6.0     17.5   17.5   17.5   17.5   Kinite   8.1   84.0   Kinite   6.0     17.5   17.5   17.5   Kinite   8.1   84.0   Kinite   6.0		. 82	714	25	16.3	Knife	8.1	84.0	Kirle	8.0	83.0	11	96	7.7	4884
864         8878         \$7         1166         Rolle         8.1         \$840         Kulle         6.0           1100         986         57         127         Kulle         8.1         \$840         Kulle         6.0           1100         986         57         120         Kulle         8.1         9840         Kulle         6.0           1239         1110         980         57         20         Kulle         6.0         6.0           1239         1110         59         27         20         Kulle         6.0         Kulle         6.0           1101         59         57         20         Kulle         8.1         84.0         Kulle         6.0           1102         1110         59         22         Kulle         8.1         84.0         Kulle         6.0           1102         1110         50         22         Kulle         8.1         84.0         Kulle         6.0           1102         110         50         23         Kulle         8.1         84.0         Kulle         6.0           1102         110         50         23         Kulle         8.1         84.0 <td>9</td> <td></td> <td>171</td> <td>2%</td> <td>15.4</td> <td>Knife</td> <td>200</td> <td>84.0</td> <td>Knife.</td> <td>6.0</td> <td>6110</td> <td>- 71</td> <td>- 80</td> <td>N</td> <td>463%</td>	9		171	2%	15.4	Knife	200	84.0	Knife.	6.0	6110	- 71	- 80	N	463%
1940   985   547   1477   Marke   811   9840   Marke   6.0     1940   987   577   1828   Marke   8.1   9840   Marke   6.0     1726   1105   577   1200   Marke   8.1   9840   Marke   6.0     1726   1105   577   27.1   Marke   8.1   9840   Marke   6.0     1417   1117   528   224   Marke   8.1   9840   Marke   6.0     1418   548   224   Marke   8.1   9840   Marke   6.0     1428   1129   528   224   Marke   8.1   9840   Marke   6.0     1439   1124   528   224   Marke   8.1   9840   Marke   6.0     1430   1124   528   224   Marke   8.1   9840   Marke   6.0     1540   1445   528   224   Marke   8.1   9840   Marke   6.0     1540   1452   529   224   Marke   8.1   9840   Marke   6.0     1540   1452   54   224   Marke   8.1   9840   Marke   6.0     1540   1450   234   Marke   8.1   9840   Marke   6.0     1540   1540   234   Marke   8.1   9840   Marke   6.0     1541   1542   234   Marke   8.1   9840   Marke   6.0     1542   1543   1544   Marke   8.1   9840   Marke   6.0     1544   1545   Marke   8.1   9840   Marke   6.0     1545   1546   Marke   8.1   9840   Marke   6.0     1546   1546   Marke   8.1   9840   Marke   6.0     1547   1548   Marke   8.1   9840   Marke   6.0     1548   1549   Marke   8.1   9840   Marke   6.0     1549   1540   Marke   8.1   9840   Marke   6.0     1540   1540   Marke   8.1   9840   Marke   6.0     1540   1540   Marke   8.1   9840   Marke   6.0     1541   1541   1541   1541   Marke   8.1   9840   Marke   6.0     1542   1544   Marke   8.1   9840   Marke   6.0     1544   1544   Marke   8.1   9840   Marke   6.0     1545   1546   Marke   8.1   9840   Marke   6.0     1546   1546   Marke   8.1   9840   Marke   6.0     1546   1546   Marke   8.1   9840   Marke   6.0     1547   1548   Marke		24	828	25	16.6	Knife	8.1	84.0	Knife	6.0	63.0	21	96	71	430%
1909   2942   577   1828   Marke   8.1   84.00   Marke   8.00     1739   11056   577   22.17   Marke   8.1   84.00   Marke   8.0     1739   11141   5.7   22.13   Marke   8.1   84.00   Marke   8.0     1747   17414   5.7   22.14   Marke   8.1   84.00   Marke   8.0     1747   17414   5.2   22.14   Marke   8.1   84.00   Marke   8.0     1748   1747   5.2   22.14   Marke   8.1   84.00   Marke   8.0     1748   1747   22.2   Marke   8.1   84.00   Marke   8.0     1749   1740   22.2   Marke   8.1   84.00   Marke   8.0     1740   1741   8.2   Marke   8.1   84.00   Marke   8.0     1741   8.0   9.0   9.0   Marke   8.1   84.00   Marke   8.0     1742   1743   8.0   9.0   Marke   8.1   84.00   Marke   8.0     1743   22.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     1744   22.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     1745   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.14   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.15   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.16   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.17   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.18   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.19   27.00   6.0   43.2   Marke   8.1   84.00   Marke   8.0     27.19   27.00   6.0   43.2   Marke   8.1   84.00   Marke   6.0     27.19   27.00   6.0   43.2   Marke   8.1   84.00   Marke   6.0     27.19   27.00   6.0   43.2   Marke   8.1   84.00   Marke   6.0     27.10   27.00   6.0   43.2   Marke   8.1   84.00   Marke   6.0     27.11   27.00   6.0   43.2   Marke   8.1   84.00   Marke   6.0     27.12   27.00   6.0   43.2   Marke   8.1   84.00   Marke   6.0     27.13   27.00   27.00   67.00   43.2   Marke   8.1   84.00   Marke   6.0     27.14   27.15   27.10	-		8885	23	17.7	Knife	8.1	84.0	Knife	6.0	63.0	7.1	96	71.	40204
1236         599         57         2000         Kolebe         8.1         94.0         Kolebe         6.0           1176         57         27.1         Kolebe         8.1         94.0         Kolebe         6.0           1177         1177         57         27.1         Kolebe         8.1         84.0         Kolebe         6.0           1177         1177         58         24.4         Kolebe         8.1         84.0         Kolebe         6.0           1178         58         24.4         Kolebe         8.1         84.0         Kolebe         6.0           1189         127         Kolebe         8.1         84.0         Kolebe         6.0           1194         58         24.4         Kolebe         8.1         84.0         Kolebe         6.0           1194         58         24.5         Kolebe         8.1         84.0         Kolebe         6.0           1194         58         24.5         Kolebe         8.1         84.0         Kolebe         6.0           1194         14.0         27.1         Kolebe         8.1         84.0         Kolebe         6.0           1195         14.0<		.00	942	15	10.8	Knife	8.1	84.0	Knife	6.0	610	7.1	96	1,1	376%
1736         1704         57         21.1         Kelete         8.1         84.0         Kelete         6.0           1177         1171         54         22.4         Kelete         8.1         84.0         Kelete         6.0           1177         1177         54         22.4         Kelete         8.1         84.0         Kelete         6.0           1504         1278         54         Kelete         8.1         84.0         Kelete         6.0           1504         1274         54         A.6         Kelete         8.1         84.0         Kelete         6.0           1604         1434         54         A.6         Kelete         8.1         84.0         Kelete         6.0           1604         1434         54         A.6         Kelete         8.1         84.0         Kelete         6.0           1604         1434         74         A.7         Kelete         8.1         84.0         Kelete         6.0           1604         1434         A.4         A.7         Kelete         8.1         84.0         Kelete         6.0           1604         1434         Kelete         8.1         84.0 <td>12</td> <td></td> <td>566</td> <td>25</td> <td>26.0</td> <td>Knife</td> <td>8.1</td> <td>84,0</td> <td>Knife</td> <td>6.0</td> <td>63.0</td> <td>7.1</td> <td>96</td> <td>7.1</td> <td>359%</td>	12		566	25	26.0	Knife	8.1	84,0	Knife	6.0	63.0	7.1	96	7.1	359%
1377   1171   559   22.3   Knife   8.1   84.0   Knife   8.0	ŀ	36	1056	157	27.7	Knife	8.1	84.0	Knife	6.0	63.0	w	90	7.1	33.7%
1472   1171   588   244   Kohre   811   580   Kohre   610	10		1114	15	27.1	Knife	8.1	840	Knife	6.0	630	T.	96	11	3199h.
1778   1789   588   246   Mohre   81   840   Mohre   610     1940   1787   558   25.4   Kohle   81   840   Kohle   610     1941   1943   558   25.4   Kohle   81   840   Kohle   610     1944   1403   559   25.4   Kohle   81   840   Kohle   610     1945   1452   59   274   Kohle   81   840   Kohle   610     1955   1454   74   77   Kohle   81   840   Kohle   610     1955   1454   74   77   Kohle   81   840   Kohle   610     1955   1454   74   77   Kohle   81   840   Kohle   610     1955   1454   74   77   74   77   Kohle   81   840   Kohle   610     1956   1454   74   77   Kohle   81   840   Kohle   610     2703   1454   77   77   Kohle   81   840   Kohle   610     2713   2700   610   420   Kohle   81   840   Kohle   610     2714   2715   610   610   610   610     2715   2700   610   610   610   610     2716   2701   610   610   610     2717   2701   610   610   610     2718   2701   610   610   610     2719   2701   610   610   610     2710   2701   610   610   610     2711   2701   610   610   610     2712   2701   610   610   610     2713   2701   610   610   610     2714   2714   610   610     2715   2701   610   610     2716   610   610     2717   2701   610   610     2718   610   610   610     2718   7401   610   610     2719   7401   610   610     2710   7401   610     2711   7401   610   610     2711   7401   7401		12	1/11	89	224	Knife	8.1	84.0	Knife	80	83.0	77	98	11	30-196
1504   1704   549   25.7   Kalie   8.1   84.0   Kalie   6.0     1945   549   549   75.4   Kalie   8.1   84.0   Kalie   6.0     1945   1453   559   754   Kalie   8.1   84.0   Kalie   6.0     1965   1454   542   754   754   754   754   754   754   754     1965   1454   754   754   754   754   754   754   754   754     1555   1454   754   754   754   754   754   754   754   754     1756   1757   175			1239	85	246	Kniře	8.1	84.0	Knife	6.0	63.6	7.1	06	- 11	289%
1945   1945   548   745.5   746.15		100	1207	5.8	757	Knife	8.1	84.0	Knife	6.0	63.0	7.1	8	71	276%
1403   549   28.1   Karde   8.1   84.0   Karde   8.0     1403   543   28.2   ZAA   Karde   8.1   84.0   Karde   8.0     1403   1453   244   23.2   Karde   8.1   84.0   Karde   8.0     1504   1454   24.2   ZAA   ZAA   ZAA   ZAA   ZAA   ZAA   ZAA     1505   1454   25.2   Karde   8.1   84.0   Karde   8.0     1504   1454   25.2   Karde   8.1   84.0   Karde   8.0     2723   1454   25.2   24.2   Karde   8.1   84.0   Karde   6.0     2724   1454   8.2   24.2   Karde   8.1   84.0   Karde   6.0     2725   2706   6.0   44.2   Karde   8.1   84.0   Karde   6.0     2726   2727   2707   6.0   44.2   Karde   8.1   84.0   Karde   6.0     2727   2708   6.0   44.2   Karde   8.1   84.0   Karde   6.0     2728   2729   2729   44.4   Karde   8.1   84.0   Karde   6.0     2729   2720   6.0   44.2   Karde   8.1   84.0   Karde   6.0     2720   2720   6.0   44.2   Karde   8.1   84.0   Karde   6.0     2721   2720   6.0   44.2   Karde   8.1   84.0   Karde   6.0     2721   2720   2720   6.1   44.1   Karde   8.1   84.0   Karde   6.0     2721   2720   2720   2720   2720   2720   2720   2720     2721   2720   2720   2720   2720   2720   2720   2720     2722   2722   2720   2720   2720   2720   2720   2720     2723   2720   2720   2720   2720   2720   2720     2724   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720     2725   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720   2720     2720   2720   2720   2720   2720			1345	- 88	26.9	Knife	8.1	84.0	Knife	6,0	63.0	7.1	30	71.	264%
1985         1492         543         23.3         Karlie         8.1         86.0         Korlie         6.0           1895         143         37.2         Karlie         8.1         86.0         Korlie         6.0           1896         1594         74         32.2         Karlie         8.1         84.0         Karlie         6.0           1896         1694         74         32.2         Karlie         8.1         84.0         Karlie         6.0           1194         72         34.2         Karlie         8.1         84.0         Karlie         6.0           210         1007         59         30.0         Karlie         8.1         84.0         Karlie         6.0           210         1007         50         50         50         50         50         6.0         6.0           2173         100         60         31.2         Karlie         8.1         80.0         Karlie         6.0           2173         2180         60         60         43.2         Karlie         8.1         80.0         Karlie         6.0           2173         2100         60         43.2         Karlie         <		544	1.603	65	28.1	Knife	8.1	84.0	Knife	6.0	93.0	1.1	96	21	25,2%
1985   1756   74   30.7   Kahle   81   84.0   Kahle   6.0			1462	.895	282	Knife	8.1	84.0	Knife	6.0	63.6	7.1	96	1/2	243W.
1564   1584   74   322   Kinle   8.1   8450   Kinle   6.0     1584   1584   74   322   Kinle   8.1   8450   Kinle   6.0     1583   1584   74   322   Kinle   8.1   8450   Kinle   6.0     2003   1083   5.9   30.0   Kinle   8.1   8450   Kinle   6.0     2003   1083   5.9   30.0   Kinle   8.1   8450   Kinle   6.0     2004   1583   859   30.0   Kinle   8.1   8450   Kinle   6.0     2005   2006   6.0   8.0   Kinle   8.1   8450   Kinle   6.0     2006   2006   4.20   Kinle   8.1   8450   Kinle   6.0     2007   2008   6.0   4.20   Kinle   8.1   8450   Kinle   6.0     2008   2009   4.2   Kinle   8.1   8450   Kinle   6.0     2009   2009   2009   2009   2009   2009   2009     2009   2009   2009   2009   2009   2009     2009   2009   2009   2009   2009   2009     2009   2009   2009   2009   2009     2009   2009   2009   2009   2009     2009   2009   2009   2009     2009   2009   2009   2009     2009   2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009   2009     2009   2009     2009   2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009   2009     2009     2009   2009     2009   2009     2009     2009   2009     2009     2009     2009   2009     20		.59	1536	7.4	30.7	Kulle	8.1	84.0	Knife	6.0	63.0	31	90	7.1	232%
1534   74   23.7   Anile   8.1   84.0   Kolle   6.0			1609	74	322	Kufe	8.1	84.0	Knife	6.0	810	71	36	7/	227%
1743   545   546	4	950	1604	74	33.7	Kulfe	9.1	84.0	Knife	6.0	63.0	71	960	7.1	211%
2103         14002         5/9         36,0         Reduce         81,1         84/0         Reduce         60           27238         1937         5/9         37.2         Kindle         81         86/0         Kindle         60           2375         1909         6/0         37.8         Kindle         8.1         86/0         Kindle         60           2375         2709         6/0         37.8         Kindle         8.1         86/0         Kindle         60           2731         2700         6/0         40.2         Kindle         8.1         86/0         Kindle         60           2847         2720         40.0         40.2         Kindle         8.1         86/0         Kindle         60           2731         2740         40.0         40.2         Kindle         8.1         84/0         Kindle         8.0           2731         2444         41.0         Kindle         8.1         84/0         Kindle         8.0           2731         2445         51.0         Kindle         8.1         84/0         Kindle         6.0           2731         2445         51.0         Kindle         8.1	-		1743	65	34.9	Kulle	8.1	94.0	Knife	0.0	63.0	7.1	96	71	20406
7228		503	1002	5.9	36.0	Knife	8,1	94.0	Knife	0.0	63.0	7,1	90	1,1	197%
7238 11971 860 1884 Knule 8.1 8440 Knule 6.0 2175 21900 660 4408 Knule 8.1 8400 Knule 6.0 22175 21900 660 450 Knule 8.1 8400 Knule 6.0 22171 21800 660 450 Knule 8.1 8400 Knule 6.0 2217 2180 660 450 Knule 8.1 8400 Knule 6.0 2217 2220 660 450 Knule 8.1 8400 Knule 6.0 2217 2220 660 450 Knule 8.1 8400 Knule 6.0 2217 2340 611 48.1 Knule 8.1 8400 Knule 6.0 2217 2465 61 48.0 Knule 8.1 8400 Knule 6.0 2217 2465 61 48.0 Knule 8.1 8400 Knule 6.0	+		1991	13	37.2	Knite	8.1	84.0	Knife	6,0	63.0	1,1	96	7.1	197%
2.375         T1980         860         3700 kinde         8.1         84.0         Kinde         6.0           75.11         2700         600         40.2         Kinde         8.1         84.0         Kinde         6.0           75.11         2700         600         44.2         Kinde         8.1         84.0         Kinde         6.0           75.11         2700         60         43.2         Kinde         8.1         84.0         Kinde         6.0           284.7         2721         60         44.4         Kinde         8.1         84.0         Kinde         6.0           284.1         60         44.4         Kinde         8.1         84.0         Kinde         6.0           284.1         60         44.4         Kinde         8.1         84.0         Kinde         6.0           272.1         60         44.5         Kinde         8.1         80.0         Kinde         6.0           272.1         60         61         46.5         Kinde         8.1         80.0         Kinde         6.0           272.1         62         62         62         Kinde         6.1         6.0         6.0		38	1551	80	38.4	Keife	8.1	84.0	Knife.	6.0	0,08	7.1	96	12	185%
2335 21040 640 4403 Kirlate 81, 84,0 Kirlite 6,0 2751 2100 640 402 Kirlate 81, 84,0 Kirlite 6,0 2751 2100 640 402 Kirlite 81, 84,0 Kirlite 6,0 2751 2200 670 444 Kirlite 81, 84,0 Kirlite 6,0 2752 2761 465 Kirlite 81, 84,0 Kirlite 6,0 2753 2449 611 48.3 Kirlite 81, 84,0 Kirlite 6,0 2753 2449 611 48.3 Kirlite 81, 84,0 Kirlite 6,0 2754 2445 61 48.3 Kirlite 81, 84,0 Kirlite 6,0 2755 2445 61 48.3 Kirlite 81, 84,0 Kirlite 6,0			1980	09	39.6	Knife	8.1	84.0	Knife	6.0	.0759	71	96	71	180%
2551 2700 640 4420 Kinle 8.1 8440 Kinle 640 2551 2720 640 4424 Kinle 8.1 8400 Kinle 640 2551 2720 640 4424 Kinle 8.1 8400 Kinle 640 2751 2751 8404 611 441 Kinle 8.1 8400 Kinle 640 2752 2744 611 441 Kinle 8.1 8400 Kinle 640 2753 2745 61 444 Kinle 8.1 8400 Kinle 640 2754 2755 2745 61 445 Kinle 8.1 8400 Kinle 640 2755 2745 61 445 Kinle 8.1 8400 Kinle 640 2756 2757 2745 61 445 Kinle 8.1 8400 Kinle 640 2757 2745 61 445 Kinle 8.1 8400 Kinle 640		512	2040	60	808	Knife	8.1	84.0	Knife	6.0	63.0	71	96	71	174%
2511 2100 600 4422 Kalie 8.11 8400 Kalie 6.0 2687 2731 600 444 Kalie 8.1 840 Kalie 6.0 2781 7843 61 448 Kalie 8.1 840 Kalie 6.0 2773 2844 61 441 Kalie 8.1 840 Kalie 6.0 2793 7465 61 48.3 Kalie 8.1 840 Kalie 6.0 2819 7465 7465 7465 7465 7465 7465 7465 7465	-		2100	0.0	45.0	Knife	8.1	84.0	Knife	0'9	63.0	11	06	1//	16996
2847 2220 640 444 Kaine 0,1 84,0 Kolle 6,0 273,1 284,1 18,0 14,4 Kolle 8,1 84,0 Kolle 6,0 273,2 284,4 6,1 44,4 Kolle 8,1 84,0 Kolle 6,0 273,2 284,4 6,1 44,1 Kolle 8,1 84,0 Kolle 6,0 273,2 284,4 6,1 44,1 Kolle 8,1 84,0 Kolle 6,0 273,2 284,5 Kil 44,5 Kolle 8,1 84,0 Kolle 6,0 273,5 284,5 Kil 44,5 Kolle 8,1 84,0 Kolle 6,0 Kolle		111	2160	09	43.2	Knife	8.1	34.0	Knife	6.0	63.0	7.1	96	77	165%
7561 2701 600 45.6 Rafe 8.1 84.0 Knite 6.0 2727 2244 61 48.1 Knite 8.1 84.0 Knite 6.0 2727 2464 61 48.1 Knite 8.1 84.0 Knite 6.0 2727 2465 61 48.3 Knite 8.1 84.0 Knite 6.0 272 2465 61 2465			2220	0.9	14.6	Knife	8.1	84.0	Knife	6.0	63.0	71	96	11	16091
2723 2244 61 48.9 Kalde 8.1 84.0 Korle 6.0 2919 7.9 7.9 7.9 7.9 7.9 7.9 7.9 7.9 7.9 7.		242	2281	09	456	Kolle	8.1	84.0	Knife	99	63.0	7.1	8	1.1	156%
2723 2404 61 48.1 Norte 8.1 84.0 Norte 6.0 5919 7845 61 48.3 Norte 8.1 84.0 Norte 6.0			2343	2	594	Knife	8.1	84.0	Knife	6.0	63.0	7.1	96	7.1	152%
2919 7465 651 45-3 Maile 8:1 84-0 Maile 60		30	2404	61	48.1	Kaffe	6.1	84.0	Knife	.09	0,03	71	96	7.0	140%
			2465	19	49.3	Knife	8.1	840	Knife	6.0	0.03	311	90	77	346%
Nove		419	V	r			¥	· ·	×	j.	i	Y	Y		
New York New A New	9		V	X		,	)		ý	140	X	,		X	X
North	8		4	£.		ý	ť	X.	X	ť	(	1	×		Y
Boota				·				(	1				-		
Noor 2					++					-		4			
			1	i i	-	1		7.		V	X,	Y	/	1	

Notes 1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 59 (EAST - WEST)

	Column Leads	Column I reads	Column Load	Caracito (296)		As-Design	ed Axial Capacity	As-Designed Axial Capacity of Bracing Connections [Kips]	ns [Kips]			(Powided by the Connections Gale)	(Powided by the Connections Colvi	
	(Cumulative)	(Cumulative)	(Increments)			West Connection			East Connection		Viest in Tension + East	East in	Minute Berl	Percentage of Req.
Roof	[dtk]	[dix]	(Kip)	[Kip]	Conn. Type	T (Kip)	C [kp]	Conn. Type	T(kip)	C [kip]	m Comp. [kip]	in Comp [kip]	14	Bracing Capacity 784%
Floor 47	13	149	24	30	7	.0	0	Header	11.0	122.5	177	1	11	370%
Floor 46		205	15	4.1		0	.0	Header	11,0	122.5	122	11	II.	267%
Floor 45	283	242	15	5.2	V	.0	0	Header	11.0	122.5	122	11	11	209%
Flasc 44		318	.95	6.4	,	- 0	.0	Header	11.0	122.5	12.7	11	10	17.5%
Floor 43	450	375	26	7.5		.0	0	Header	No info	No info	Noinfo	No info	No info	
Floor #2	5	431	- 26	86	×	0	.0	Header	11.0	122.5	122	11	TI.	12.7%
Fluor 41	999	487	98	235		.0	0	Header	No info	No info	No info	Notinfo	Noinfo	
Fhan 40		544	99	10.9		0	0	Header	11,0	122.5	122	11	11	101%
Floor 39	680	109	25	12.0		.0	0	Header	No info	No info	Nomo	Notinto	No info	
F100t 38		259	95	12.1	,	. 0	0	Header	11.0	122.5	122	111	11	2000
Floor 37	828	114	21	14.3		0	0	Header	11.0	122.5	221	11	11	1600
Floor 36		m	57	154	C	- 0	0	Header	11.0	122.5	122	- 11	11	4300
Floor 35	964	828	25	16.6	í	0	0	Header	0.11	122.5	122		11	1993
Floor 34		5988	25	17.7.		.0	0	Header	11.0	122.5	122	- 11	W.	62%
Floor 33	1100	245	257	16.8	1.0	0	0	Header	0,11	1725	132	11	11	5894
Floor 32		. 666	57	20.0		. 0	.0	Header	11,0	172.5	132	11	П	59%
Floor 31	1236	9501	25	21.1		. 0	.0	Heater	11.0	122.5	122	11	11	57%
Floor 30		1114	25	22.3	,	- 0	0	Header	11.0	(22.5	17.7	11	11	484
Floor 29	1372	1171	58	23.4		.0	.0	Header	31,0	122.5	122	- 11	11	4705
Floor 28		1229	58	24.6	į	0	0	Header	11.0	122.5	122	TI.	11	450
Fluor 27	1508	1287	88	25.7	,	.0	0	Heade	11.0	122.5	17.1	11	11	43.00
Fluor 26		1345	88	589		0.	0	Header	11,0	122.5	122	- 11	111	40.60
Floor 25	1644	1403	58	28.1		0	.0	Header	11.0	122.5	122	11	TI.	1000
floor 24		1462	28	292	,	0	0	Header	11.0	122.5	122	11	=	38%
Hoyr 23	1865	1536	74	30.7		o	0	Header	11.0	122.5	122	=	=	*60
Floot 72		1693	74	37.7	-	. 0	0	Header	13.0	177.5	122	=	2	34.46
Floor 21	1950	16094	74	33.7	×	0	0	Header	0.0	122.5	122	- D	=	134
Floor 20		1743	59	34.9	x	0	0	Header	11.0	122.5	122	- 10	=	320
Floor 19	2103	1902	59	36.0	ı	0	0	Header	11.0	177.5	122	н	H	1001
Hoor 18		1881	55	37.7		0	.0	Header	11.0	122.5	122		=	Mode
Floor 17	3239	1651	60	384		0	0	Header	11.0	122.5	123	=	=	305%
Flaor 16		1980	69	39.6		.0	0	Header	11.0	122.5	122	11	11	7887
Hoor 15	2375	2040	99	40.8	ı	0	0	Header	11.0	122.5	122	11	=	27%
Floor 14		2100	99	42.0	1	.0	0	Header	11.0	122.5	122		- 11	5662
Floor 13	2511	2160	60	432	3.0	0	ø	Header	11.0	122.5	1221	D	11	- 62
Floor 12		2220	99	44.4	*	0	0	Header	11.0	122.5	122	II.	0	2504
Floor II	2647	7281	09	45.6	,	0	0	Header	11.0	177.5	122			24%
Hoor 10		2343	3	46.9	,	0	0	Header	11.0	4225	122	11		23%
Hoor 9	2733	2404	61	40.1		0	0	Header	11.0	122.5	122			27%
Hoer 8		2465	61	49.3		0	0	Header	11.0	1225	122	11	-13	822
Hoor 7.	2919	,				7		,		1		-	7	
Hoor 6		-	9			y.	7	7	Y	7			,	
Hour 5			Ž.	,	ţ-i	,		1	-	í	r.	2		, c
Hoor 4						0								
Floor 3		,	X		,	Y	1	,	ý	X	y	,	1	,
Hose 2		-												

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Roor	As built Axial Column brads.	Design Axial Column Logils	Calimin Loads	Capacity (29th)		As-Desig	ned Axial Capacity	As-Designed Axial Capacity of Bracing Connections [Kips]	is [Kips]			(Provided by the Connections Only)	Sunections Only)	
	(Camulative)	(Cumulative)	(Camalative)			North Connection			South Connection	1	North in Tension +	South in Tension +	Minimum (bin)	Percentage of Req
	lkpl	[Kp]	(Kp)	(Kip)	Conn Type	Thiel	C Bipl	Conn Type	This	Citipl	South in Comp. [ktp]	Rorth in Comp. (top)	Management (App)	Bracing Capacity
Road		- 23	Z	50	Knife	8.1	840	Kutte	80	63.0	73	9	11	15210%
Flast 47	11	75	25	1.5	Knife	6.0	63.0	Knife	1.8	847	36	11	12	4732%
Floor 46		126	15	25.	Knife	6,0	63.0	Kalfe	8.1	84.0	06	11	71	281990
Hoor 45	311	105	23	3.7	Knife	6,0	63.0	Kuite	8.1	84.0	90	71	7.1	191016
Floor 44		246	65	4.9	Knife	6.0	63.0	Knife	8.1	84.0	96	E	71	1453%
Fluor 43	794	304	-65	9.6.0	Knife	6.0	63.0	Kuife	8.1	84.0	30	71	111	117,0%
Fluor 42		363	-65	7.3	Knife	6,0	63.0	Knife	- 81	84.0	96	71	71	97906
Floor 41	612	423	65	58	Knife	6.0	63.0	Knife	8.1	84.0	0.6	, u	77	R419b
Floor 40		482	600	9.6	Knife	60	63.0	Knife	8.1	840	30	7.1	7.1	73780
Flanc 39	767	542	0.9	10.8	Knife	6.0	63.0	Knife	8.1	84.0	30	7.1	71	6569W
Hoor 38		209	0.9	12.0	Knife	6.0	63.0	Knife	8.1	84.0	90	И	1/	90069
Floor 37	912	288	00	10.2	Knife	8.0	63.0	Kinte	8.1	84.0	96	K	74	元05
Floor 36		123	19	14.5	Knife	6.0	63.0	Knife	1.3	84.0	36	77	N	419248
Fluor 35	1062	784	19	15.7	Knife	6.0	63.0	Knife	8.1	84.0	96	71	71	454%
Floor 34		.845	19	169	Knife	.0'9	63.0	Knife	8.1	84.0	96	71	71.	42196
Floor 13	1212	506	1.0	10.1	Kaife	99	630	Knife	8.1	84.0	06	11	7.1	39306
Floor 32		956	19	19.3	Knife	6.0	63.0	Knife	8.1	84.0	96	7.1	7.1	369%
Floys 31	1362	1028	19	20.6	Knife	6,0	63.0	Knife	8.1	84.0	30	- 12	71	340%
Floor 30		1009	29	218	Knife	6.0	630	Knife	8.1	84.0	90	11	11	3278
Floor 29	1512	11511	67	23.0	Knife	8.0	83.0	Knife	8.1	84.0	90	71	71	309%
Fluor 28		1213	62	243	Knife	6.0	63.0	Knife	8.1	84.0	96	71	- 11	293%
Eleoc 22	1662	1275	. 63	755	Knife	6.0	63.0	Knife	8.1	84.0	96	7.1	11	279%
Hoor 26	4	1337	29	26.7	Knife	6,0	63.0	Knife	8.1	84.0	90	r.	71	268%
Floor 25	1812	1398	-63-	28.0	Knite	6.0	63.0	Knife	8.1	84.0	96	7.7	71	254%
Flace 24		1991	29	292	Knife	6.0	63.0	Knife	8.1	84.0	30	73	7.1	2458
Fluor 23	1962	1540	79	308	Knife	6.0	63.0	Knife	8.1	840	90	7.1	7.1	2310m
Floor 22		1619	80	32.4	Kuffe	6.0	84.0	Kuife	8.1	84.0	06	E.	71	220%
Floor 21	2150	1699	600	34.0	Kulfe	60	63.0	Knife	8.1	84.0	360	71	71	2050k
Floor 20		1763	5	353	Kulle	00	63.0	Kulfe	8.1	94.0	96	7.1	11	20208
Hoor 19	2319	1027	64	36.5	Knite	6,0	03.0	Knife	8.1	040	90	E.	7.7	195/8
Hoor 18		1691	69	3/8	Knife	6.0	63.0	Knife	8.1	84.0	96	17	41	1887%
Flant 17.	2469	1954	64	38.1	Knite	60	019	Knife	8.1	84.0	96	17	7.1	18296
Fluor 16		2018	64	40.4	Knife	6.0	63.0	Knife	8.1	84.0	90	TJ.	71	1786
Hoor 15	2619	2083	64	41.7	Knife	6.0	630	Knife	8.1	84.0	.06	7.1	7.1	17300
Roor 14		2147	65	429	Knife	60	63.0	Knife	8.1	84.0	90	71	71	166%
Floor 13	2369	2212	65	44.2	Knife	6.0	63.0	Knife	8.1	84.0	- 30	71	71.	161%
Hear 12		2277	.65	45.5	Knife	6,0	63.0	Knife	8.1	84.0	96	71	11	150%
fleor 11	2919	2342	65	46.8	Koife	6.0	63.0	Knife	8.1	840	- 06	7	7.1	1529s
Floor 10		2408	67	482	Knife	60	63.0	Knife	8.1	84.0	06	7.1	71.	146%
Floor 9	3069	2473	999	49.5	Kaite	6.0	63.0	Knife	8.1	64.0	- 36	71	2.0	344%
Floor B		2539	.99	50.8	Knife	6.0	010	Kaife	8.1	84.0	90	a)	71.	14006
Floor 7	3234	2678	139	51.6	Knite	6.0	63.0	Knife	12.4	126.0	132	75	75	14196.
Hoore		2721	43	564	Knife	8.1	94.6	Knife	8.1	84.0	. 92	85	25	169%
Hoor 5	3487	2883	167	57.1	Knite	103	105.0	Knife	11.2	168.0	178	116	116	20106
Floor 4		2988	105	808	Knile	8.1	840	Knife	103	106.0	113	3	H	158%
Boor 3	3850	2174	1106	63.5	Kuffe	1439	147.0	Knife	181	0.681	204	180	166	262%
*******														

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**Guy Nordenson and Associates** 

INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 60 (EAST - WEST)

Floor	Column brads	Column Loads	Column Loads	Capacity (29ki		As-Desig.	ned Axial Capacity	As-Designed Axial Capacity of Bracing Connections [Kips]	ons [Kips]			(Provided by the Connections Only)	onnections Only	
	(Cumulative)	(Cumulative)	(Cumulative)			West Connection			East Connection	1	Viest in Tension + East	Eastin	Minimum (Bin)	Percentage of Req.
Real	Repl	[Kip]	[Kp]	(Kip)	Conn Type Header	T [Kip]	C [6p]	Conn. Type Header	7 [kip] 10.7	C [66]	in Comp. [kip]	in Comp. [kip] 138	136	Bracing Capacity 29071%
Floor 47	11	75	25	1.5		0	0	Header	10.7	122.5	17.7	- 41	- 11	711%
Floor 46		126.	51	25.		00	0	Header	16.3	6,181	10.2	91	91	64408
Hoor 45	311	105	65	3.7		.0	0	Header	10,7	129.9	130	11	11	28.6%
Floor 44		245	65	4.9	X	0	.0	Header	10.7	129.9	130	11	ti.	218%
Floor 43	794	304	53	6.1	1	0	:0	Header	10.7	122.5	122	-11	11	176%
Floor 52		363	- 65	7.3		0	.0	Header	10.7	122.5	122	11	- 11	147/6
Fluor 41	612	423	65	5.8		0	0	Header	10.7	122.5	177	H	11	126%
Floor 40		482	08	96.		-0	0	Header	10.7	122.5	122	-11	11	31196
Fluor 39	292	542	0.9	10.8		. 0	0	Header	10.7	122.5	122	11	41	SINDS
Hoor 38		502	09	12.0		0	0	Header	10.7	122.5	122	11	11	69%
Floor 37	912	. 682	.09	13.2	,	0	.0	Header	10.7	122.5	12.2	11	11	48144
Floor 36		123	15	14.5		0	0	Header	10.7	172.5	122	11	11	All C
Fluor 35.	1067	784	19	15.7	T.	.0	.0	Header	10.7	122.5	17.7	- 11	11	1989
Floor 34		845	19	169		0	.0	Header	10.7	122.5	122		11	WEEN!
Floor 33	1212	506	19	18.1	,	0	0	Header	10.7	122.5	122	11	11	860%
Floor 32		996	19	19.3	3	0	0	Header	10.7	122.5	122	- 11	11	200
Floor 31	1362	1028	19	20.6		.0	0	Header	10,7	122.5	122	11	- 11	2000
Floor 30		1009	29	218		0	0	Header	10,7	172.5	17.7	11	11	West .
Floor 29	1512	1151	67	23.0	X	0	.0	Header	10.7	172.5	122	. 11	11	469%
Floor 28		1213	62	24.3	y	.0	0	Header	10.7	122.5	122	11	- 11	9406
Floor 27	7991	1275	29	25.5		0	0	Header	10.7	122.5	122	11	11	A298
Hoor 26		1337	29	26.7	3	.0	0	Header	10,7	122.5	122	11	=	4004
Floor 25	1812	1338	29	28.0		0	0	Header	10.7	122.5	122	11	13	1894
Floor 24		1461	62	292	1	0	0	Header	10.7	172.5	122	11	11	370%
Fluor 23	1962	1540	79	30.8		.0	0	Header	10.7	122.5	122	11	11.	USA:
Floor 22	100	1619	00	32.4		0	0	Header	10,7	122.5	122	= :	-	767
Floor 21	2150	1699	900	34.0		Ø.	0	Header	10.7	122.5	122			3 100
Flast 20		1763	5	353		0	0	Header	107	122.5	122			300
Hoor 19	2319	1027	64	365		0	0	Header	10,7	122.5	177	11	11	7,500
Hoor 18	-	1691	100	37.8		0	0	Header	107	172.5	122		= :	2360
FROM 14	7,469	1994		238.1		0	0 0	Header	10.7	4221	771		= :	7477
Floor 15	9630	2000	04	41.7		0		Header	10.7	122.5	100			part.
Floor 14	20.02	2147	42	42.9		0	0	Header	10.7	1225	19.2		=	2504
Floor 13	2763	2213	65	44.2		0	0	Header	10.7	122.5	122		-	249,
floor 12		2277	69	45.5		.0	0	Header	10.7	122.5	122	1	11	2.0%
Floor 11	2919	2342	685	46.8		.0	0	Header	10.7	122.5	122	11	111	2.0%
Floor 10		2408	67	482	1	0	0	Header	10.7	172.5	122	11	11	3778
Floor 9	3069	2473	98	49.5		0	0	Header	10.7	122.5	122	- 11	110	23%
Roor 8		2539	99	50.8		0.	0	Header	10.7	122.5	122	11	- 10	270%
Hoor 7	3234	2678	139	53.6		0	0	Header	11,6	129.9	130	12.	12	750%
Hoor 6		2721	43	54.4	Header	52	100.2	Header	12.0	143.8	151	112	.112	20696
Hoor 5	3487	2883	167	57.7	,	0	0	Header	69.5	350.8	351	0/	70	121%
Hoor 4		2,688	108	868		0	0	Header	Notato	No late	No info	Noinfo	No info	
Hoor 3	DSM.	2174	180	67.5		0	0	Header	40.4	787.1	282	40	40	0/4/4
Floor 2			444	2 10 10		-		111-11-11	Cont.	Same.	144	46	300	-

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Common Name         Common Name         Free Name         Send Connection         Page 1         Control of	Hoor As Es	As built Axial Column brads	Design Axtal Polumn Inads	Design Axial Column I read	Reg Donn Axial Canacito Driki		As Design	ned Axial Capacity o	As Designed Axial Capacity of Bracing Connections [Kips]	ns [Kips]			Total Bracing Capacity in the North-South Direction (Provided by the Connections Calid)	he North-South Direction	
1941   1944		(Cumulative)	(Cumulative)	(Increments)		A STATE OF	North Connection	1000		South Connection	1000	North in Tension +	South in Tension +	Minimum (Rip)	Percentage of Req
14   14   15   15   15   15   15   15		Ropi	Kipl	(Kip)	(Kp)	Conn type	Thip	Pikipi	Conn. Type	T Displ	P [kip]	South in Comp. (ktp)	North in Comp. (tip)		Bracing Capacity
1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1,	Honel	1	213	213	43	Knife	**	840	Knife	8.1	850	2.5	7.6	267	2167%
666         6140         6180         6140         6180         6141         6180         6140         6180         6141         6140	15 XX	138	383	140	2	Kuife	60	63.0	Knife	83	84.0	30		17	100898
10.00   10.0	loor At	COL	455	100	100	Scatco	45.4	42	Knite	0.1	0.40	100	20	9 5	46309
11   11   11   11   11   11   11   1	loor 44	200	828	6.0	12.6	Control	42	42	Kolle	6.1	94.0	126	200	8 5	40000
1971   1975	Sant 43	810	210	All and a second	14.2	Scaled	42	42	Kolfe	21	840	176	95	8 8	35.9%
1914         915         917         Sabert         92         9404         917         9104         917         9104         917         9104	Fluor 62		792	63	15.8	Seated	42	45	Knife	8.1	84.0	126	99	95	316%
1417         182         182         Saled         42         Notified         81         810         182         98         98         182         98         98         98         182         98         98         182         98 <td>Floor 41</td> <td>1014</td> <td>875</td> <td>83</td> <td>17.5</td> <td>Seated</td> <td>42</td> <td>42</td> <td>Knife</td> <td>8.1</td> <td>84.0</td> <td>126</td> <td>95</td> <td>05</td> <td>2869h</td>	Floor 41	1014	875	83	17.5	Seated	42	42	Knife	8.1	84.0	126	95	05	2869h
U101         1014         III D         2014         Samed         Q         Number         81         8464         1785         969         99         99           1472         1792         102         273         Samed         47         40         Minde         811         8460         1786         99         99           1472         1793         1871         Samed         47         40         Minde         811         8460         1786         99         99           1878         1778         1871         Samed         47         40         Minde         811         8460         1786         99         99         99           1879         1871         1871         8741         Samed         47         1874         810         1786         99	loor 40		856	82	19.2	Seated	42	42	Knife	1.8	84.0	128	05	06	262%
447         172         GT         23.3         Sented         QT         GNICE         81.1         86.0         175         SS         SS         SS         AS	185 3G	1218	1041	113	20.8	Sealed	42	42	Knife	8.1	84.0	136	95	05	2410%
4772         17008         878         2481         Sceled         479         470         Kindre         811         8140         170         90         90         90         90           1977         1173         811         2243         Sceled         472         427         Kindre         811         810         170         90         90           1170         1173         814         2243         Sceled         42         Kindre         811         840         170         90         90           1170         1758         814         2243         Sceled         42         42         Kindre         811         840         170         90         90         90           1170         1758         814         2344         Sceled         42         42         Kindre         811         840         170         90	OCH 38		1123	20	72.5	Seated	42	4	Knife	8.1	54.0	126	98	99	223%
1779   1779   177   278   School 472   472   674   6	18 June	1422	1208	83	24.1	Scatcul	42	24	Kinte	8.1	84.0	128	96	8	20896
With         1171         81         224         Souther         42         Guide         81         840         179         90         90         90           1100         1528         84         284         42         42         Guide         81         840         179         90         90           1100         1528         84         284         62         42         Guide         81         840         179         90         90           1800         1538         84         324         Seared         42         60         81         840         179         90         90           7848         1787         84         284         60         179         90         90         90           7848         1787         84         284         60         179         90         90         90         90           7849         1787         84         42         60         81         840         179         90         90         90         90         90         90         90         90         90         90         90         90         90         90         90         90         90 <td>SE 100</td> <td></td> <td>1288</td> <td>87.</td> <td>25.8</td> <td>Seated</td> <td>42</td> <td>42</td> <td>Knife</td> <td>8.3</td> <td>840</td> <td>128</td> <td>90</td> <td>95</td> <td>116916</td>	SE 100		1288	87.	25.8	Seated	42	42	Knife	8.3	840	128	90	95	116916
1454   1454   145   14	last 35.	1676	1371	83	27.4	Seated	42	42	Knife	8.1	84.0	126	909	95	183%
12.24   14.24   24.44   24.2	Floor 34		1454	63	29.1	Seated	45	. 42	Knife	8.1	84.0	126	96	28	172%
7094         1765         644         Sanded         427         840         187         186         187         860         175         89         99         99           7784         1778         644         34,34         Sanded         472         472         644         81         880         175         99         99           7784         1778         84         34,44         Sanded         472         644         81         880         175         99         99           7784         1778         84         34,44         Sanded         472         644         81         880         175         99         99           7784         1778         84         34,44         84         40.8         87         860         175         89         99         99           7784         1778         84         40.8         87         84         86         175         89         99         99         99           7785         84         42         84         84         84         84         89         178         89         99         99         99           7896         84         42 </td <td>Floor 33</td> <td>1830</td> <td>1538</td> <td>B4</td> <td>30.8</td> <td>Sented</td> <td>42</td> <td>42</td> <td>Knife</td> <td>8.1</td> <td>84.0</td> <td>126</td> <td>06</td> <td>98</td> <td>163%</td>	Floor 33	1830	1538	B4	30.8	Sented	42	42	Knife	8.1	84.0	126	06	98	163%
7798         1779         94         34.1         Scaled         42         8,04         61         176         90         90         90           7798         1887         84         1788         84         178         840         178         90         90         90           7798         1887         84         188         84         178         80         190         90         90           2440         2040         184         34.3         Scaled         42         42         60         178         80         190         90         90           2440         2040         184         43.3         Scaled         42         42         606         176         80         90         90         90           2440         2040         184         43.3         Scaled         42         42         606         176         176         90 <td< td=""><td>loor 32</td><td></td><td>1621</td><td>113</td><td>32.4</td><td>Seated</td><td>42</td><td>42</td><td>Knife</td><td>8.1</td><td>84.0</td><td>126</td><td>60</td><td>05</td><td>15576</td></td<>	loor 32		1621	113	32.4	Seated	42	42	Knife	8.1	84.0	126	60	05	15576
778.98         1778         878         Sanotel         472         676         676         871         8840         1776         970         970           7449         1566         84         384         Sanotel         472         676         81         8840         1776         90         90           7440         1866         84         384         Sanotel         472         676         81         8840         1776         90         90         90           7440         2794         84         442         Sanotel         472         676         81         860         176         90         90         90           7890         2794         84         442         Sanotel         472         640         81         860         176         90 <td>leor 31</td> <td>7034</td> <td>1705</td> <td>- 9:4</td> <td>34.7</td> <td>Stated</td> <td>42</td> <td>42</td> <td>Koife</td> <td>8.1</td> <td>84.0</td> <td>126</td> <td>979</td> <td>06</td> <td>147%</td>	leor 31	7034	1705	- 9:4	34.7	Stated	42	42	Koife	8.1	84.0	126	979	06	147%
7986         1887         884         33.4         Seared         472         673         Ruffe         81         840         179         60         90           2440         2796         194         33.4         Seared         42         Ruffe         81         840         176         50         90         90           2440         2796         194         44.3         Seared         42         Ruffe         81         840         176         50         90         90           2796         2796         194         44.3         Seared         42         Ruffe         81         840         176         90         90         90           2796         2797         194         45.2         Seared         42         80         176         90	00 too		1788	U.S	358	Seated	45	47	Knife	8.1	84.0	126	- 05	90	1400h
2449         11964         844         3181         Scanel         42         87         Mine         81         84.0         170         90         90           2749         2744         1844         443         Scanel         47         47         Mine         81         84.0         170         90         90           2789         2784         184         443         Scanel         47         47         67         87         90         90         90           2789         1884         443         Scanel         47         47         67         80         70         90         90         90           2789         1884         443         Scanel         47         42         80         81         80         70         90 <td>oor 29</td> <td>2758</td> <td>1872</td> <td>84</td> <td>374</td> <td>Sealed</td> <td>42</td> <td>42</td> <td>Knife</td> <td>8.1</td> <td>840</td> <td>128</td> <td>90</td> <td>90</td> <td>134%</td>	oor 29	2758	1872	84	374	Sealed	42	42	Knife	8.1	840	128	90	90	134%
2449         27940         194         44.8         Sametel         42         42         42         100         Minte         81         84.0         176         50         50         50           2949         2786         194         44.2         Sented         47         42         80         176         50         50         50           2860         2786         194         44.2         Sented         42         42         44         81         840         176         50         50         50           2860         2867         195         44         45         80         176         50         50         50           2860         2867         195         44         80         176         50         50         50           2860         2867         195         44         80         176         50 <td>sor 28</td> <td></td> <td>1956</td> <td>84</td> <td>38.1</td> <td>Seated</td> <td>42</td> <td>42</td> <td>Knife</td> <td>8.1</td> <td>84.0</td> <td>126</td> <td>20</td> <td>95</td> <td>128%</td>	sor 28		1956	84	38.1	Seated	42	42	Knife	8.1	84.0	126	20	95	128%
7946         7174         84         42.5         Scared         47.2         42.0         Knife         81         48.0         170         50         50         50           7870         2720         184         44.5         Scared         47.2         42.0         Knife         81         86.0         170         50         50         50           7870         7870         18.0         47.5         52.0         62         62         50	12 100	2442	2040	0.4	40.8	Seated	42	42	Knife	8.1	84.0	126	20	8	123%
2896         27206         194         4842         Scaled         472         472         Mine         81         640         170         50         50         50           2890         2780         1884         459         Scaled         47         43         Kinfe         81         640         176         50         50         50           2890         2891         Scaled         47         43         Kinfe         81         640         176         50         50         50           3894         2894         47         44         Kinfe         81         640         176         50         50         50           3894         2894         47         44         44         640         176         50         50         50         50           3894         2894         48         Kinfe         81         640         176         50         <	oor 26		2124	0.4	42.5	Seated	.42	45	Knife	8.1	84.0	126	. 90	50	11896
2529         184         458         Sajetel         42         Kalfe         81         6440         1755         50         50           25604         25237         185         478         Sajetel         42         42         Kalfe         81         640         1756         50         50           2564         257         50         50         50         50         50         50         50           2564         257         50         50         50         50         50         50         50           310         27/4         44         55         50         50         50         50         50         50           311         27/4         44         55         50         42         60         70         70         5	loor 25	2640	2268	84	44.2	Seated.	42	42	Knife	8.1	84.0	126	95	95	3.1499
2880         7387         105         54.79         Scaled         42         42         Kinte         811         84.0         178         50         50           1064         2531         Scaled         42         Kinte         811         84.0         178         50         50           210         2600         541         Scaled         42         Kinte         811         84.0         178         50         50           210         2600         641         Scaled         42         Kinte         811         84.0         178         50         50           210         210         Acatel         42         Kinte         811         84.0         178         50         50           210         210         Scaled         42         Kinte         81         84.0         178         50         50           210         210         Scaled         42         Kinte         81         84.0         178         50         50           210         210         Scaled         42         Kinte         81         84.0         178         50         50           210         210         Scaled	100 Z 4		2522	84	45.8	Seated	42	42	Knife	8.1	84.0	126	95	S	10948.
1,10,	1834 Z.3	2850	7397	105	613	Sealed	42	42	Knife	8.1	84.0	126	60	20	105%
Nick   7507   1465   524   524   524   442   442   444   644   1745   520   550   550	00177		2501	105	9.05	Seated	42	47	Knife	8.1	84.0	126	35	35	100%
1716   1780	00/21	3094	7607	105	52.1	Sexted	43	43	Knife	8.1	84,0	126	33	98	OSM.
11/10   11/1	183r 20		2650	64	53.8	Sented	42	45	Knife	8.1	94.0	126	20	8	1000
1577   77896   818   8	60 100	3318	2774	94	55.5	Scaled	42	45	Knife	8.1	54.0	126	8	20	9000
1557   7146   86	81 100		2859	5.0	57.2	Seated	-24	42	Knife	8.1	84.0	126	50	9	8,650
3756         116         186         1816         Scannel         42         42         Kinfe         81         84.0         1745         50         50           2756         3750         186         64.0         Scarled         42         42         Kinfe         81         84.0         1745         50         50         50           7890         2780         Scarled         42         42         Kinfe         81         84.0         175         50         50         50           7890         2780         Scarled         42         42         Kinfe         81         84.0         175         50         50         50         50           4134         5450         18         64         175         84         175         50	CON 17.	3522	2845	88	583	Seated	42	42	Knife	8.1	84.0	128	99	20	9660
3756         3146         life         life <th< td=""><td>31 von</td><td></td><td>3030</td><td>312</td><td>9709</td><td>Seated.</td><td>42</td><td>42</td><td>Knife</td><td>8.1</td><td>84.0</td><td>176</td><td>90</td><td>08</td><td>830</td></th<>	31 von		3030	312	9709	Seated.	42	42	Knife	8.1	84.0	176	90	08	830
7930         31301         815         6440         Scared         42         49         Kinfe         811         8440         175         59         50         50           7930         3330         865         652.8         Scared         42         42         Kinfe         811         840         175         59         59         50           4134         5450         137         86         17         640         175         59         59         59         50         5	90f 15	3776	3116	99	62.3	Seated	42	47	Knife	8.1	84.0	126	90	95	2008
7830         3373         86         67.5         Scaled         42         42         Rafe         81         64.0         175         59         59           4134         3479         65         67.5         Scaled         42         42         6.0fc         81         64.0         175         59         59           4134         3490         66         77         50         42         6.0fc         81         64.0         175         59         59           4318         3450         66         77         6.0fc         81         66.0         176         59         59           4218         3454         66         77         6.0fc         81         66.0         176         59         59           4218         3454         74<	00i 14		3201	035	64.0	Seated	42	45	Knife	8.1	84.0	126	50	50	10%
4134 3457 65 675 Secret 42 67 676 61 61 640 175 59 99 99 99 99 99 99 99 99 99 99 99 99	oor 13:	3830	3268	96	65.8	Seated	42	45	Knife	8.1	84.0	126	99	8	yen
4134 3459 846 85 872 Scaled 472 472 Knife 811 8450 1726 559 559 670 670 670 670 670 670 670 670 670 670	(a)r 12		3373	902	67.5	Seated	42	42	Knife	8.1	84.0	126	8	8	9.00
4739   31543   164   70.55   Seated   42   42   42   Mark   811   8440   1746   549   540   54	11 too	4134	3459	90	563	Seated	42	42	Knife	8.1	84.0	126	06	06	din.
4339 31873 86 372 5 Seated 42 42 40 Nafe 81 8040 1796 59 59 59 60 60 60 60 60 60 60 60 60 60 60 60 60	oor 10		3543	RA	70.9	Seated	.42	42.	Knife	8.1	84.0	126	05	6	914
3715   156   156   175	foor 9	4338	3629	908	72.6	Seatest	45	42	Knife	8.1	84.0	126	05	98	No.5
49-7 2399 1396 47-0 Korle 85.0 689-2 Iros-Cox NC	loor B		3715	115	243	Knife	42	42.	Knife	8.1	84.0	126	90	80	6700
4475 959 -41 1927 Kule 8.1 846 Insscor NC	100e 7	4247	2349	1166	47.0	Knite	59.0	692.0	Truss Cnx	NC	NC	NC	MC	NC	>100
4475 959 41 197 Enfe 8.1 840 Inss Oct NC	3 Joor 6		1001	-1349	20.0	Knife	8.1	84.0	Truss Cnx	NC	NC	NC	NC	NC	>100
150 1084 124 217 103 1060 Ende 6.0 639 23 111 73 13	foor 5	4475	656	-41	19.7	Knite	8.1	84.0	Truss Croc	NC	MC	NC	MC	NC	>100
190	Hoos 4		1084	124	217	х	10.3	105.0	Knife	6.0	630	73	111	73	33896
	Book 3	150		ť		5	í			(m)	*	4			- + "
	Floor 2		1	X.			Ų	X	X	Y	X	Y	1	Ť	1

Notes 1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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Floor	Colomo Lunda	Column Leads	Column lood	Physical Pills		THE PART AND	finded consequences for the first and a state of the consequences		The state of the s			(December of the Physics of Company of the Party of	Santabath Comp. Profesh	
	(Camalative)	(Cumulative)	(Increments)	(Kp)	Conn. Type	West Connection T Dopl	P  kip	Conn Pype	East Connection 7 (kip)	P (Rip)	West in Tension + East in Comp. [kip]	East in Tension a West in Comp (kip)	Minimum (Rip)	Percentage of Reg Bracing Capacity
Roof		213	213	4.3	Header	207	144.8	Heater	Notato	Marion	Noinfo	Nouro	No info	
Flax 47	138	383	140	7.3	Header	7557	1707	Header	43.5	241.3	292	214	214	3034%
Floor 46		459	901	9.2	Header	37.3	213.4	Header	68.0	322.9	360	301	301	328546
Hoor 45	500	543	04	10.9	Header	36.2	200.8	Header	16.4	122.5	199	225	159	146196
Floor 44		626	83	125	Header	164	122.5	Header	16.4	122.5	139	139	.139	1109Ns
Elsor 43	810	710	103	14.2	Header	362	2008	Header	164	122.5	159	725	159	1112%
Flanc 42		262	0.5	15.8	Header	16.4	1225.	Header	16.4	122.5	139	139	139	877%
Ficor 41	1014	875	101	17.5	Header	382	2003	Header	16.4	122.5	169	225	159	WE906
Ekor 40		356	125	19.2	Header	16.4	1725	Header	16.4	122.5	139	131	139	725/4
Elizor 39	1218	1041	83	20.8	Header	362	2003	Header	16.4	122.5	15.0	225	159	76296
Hoor 38		1123	23	22.5	Header	16.4	122,5	Header	16.4	1225	139	139	139	61916
Floor 37	1422	1206	63	24.1	Header	362	2088	Header	184	122.5	159	275	159	658%
Floor 35		1288	11.5	75.85	Header	164	177.5	Header	164	122.5	138	109	.439	5/39/6
Flant 35	1676	1371	83	22.4	Header	36.2	2083	Header	16.4	1225	159	225	159	579%
Floor 34		55年	03	29.1	Header	16.4	1225.	Header	16.4	122.5	139	139	139	47896
Floor 33	1830	1538	R4	30.8	Header	36.2	208.8	Header	16.4	122.5	158	225	159	516%
Floor 32		1621	113	32.4	Header	16.4	122.5	Header	164	122.5	139	139	139	429%
Floor 31	2034	1705	204	34.1	Header	36.7	2003	Header	16.4	122.5	159	225	159	4657m
Floor 30		1788	U.S	358	Header	16.4	172.5	Header	16.4	1725	139	170	139	31609th.
loor 29	2758	1872	84	37.4	Header	362	708.8:	Header	164	177.5	169	72%	159	424%
Floor 28		1956	84	38.1	Header	164	1225	Header	16.4	122.5	139	138	(38	355%
Elsor 27	2442	2040	0.4	40.8	Header	36.7	2003	Header .	16.4	122.5	150	225	159	38906
Hoor 26		2124	0.4	42.5	Header	16.4	1225	Header	16.4	122.5	139	139	139	327%
floor 25	2646	2268	184	.442	Header	362	208.8	Header	16.4	122.5	159	225	159	359%
Flace 24		2622	84	45.8	Header	16.4	1225	Header	16.4	1225	139	139	139	3058
Flant 23	2850	2397	105	47.9	Header	36.2	2063:	Header	16.4	1225	159	225	159	33106
Floor 22	-	1052	105	0.05	Header	16.4	1725-	Header	16.4	1775	139	139	139	278%
Floor 21	3094	7607	105	52.1	Header	36.2	2000	Header	16.4	122.5	150	325	159	304%
Floor 20		2690	0.4	518	Header	16.4	122.5	Header	164	122.5	139	100	139	25,0%
Hoor 19	3318	2774	914	525	Header	367	2003	Header	16,4	1725.	159	.775	159	20006
Floor 18		2859	90	57.2	Header	16.4	172.5	Header	364	722.5	139	139	139	263%
Flant 17	3522	2948	88	685	Header	362	208.8	Header	16.4	122.5	159	325	159	269%
Fluor 16		3630	9.8	909	Header	16.4	5221	Header	16,4	127.5	139	1379	139	229%
Hoor 15	3776	3116	90	62.3	Header	36.2	2008.8	Header	16.4	122.5	153	225	159	2559a.
Floor 14		3201	65	64.0	Header	16.4	1225.	Header	16.4	1225	139	139	139	217%
Floor 13	3930	3268	96	65.0	Reader	36.2	2003	Header	16.4	122.5	159	225	159	241%
Hear 12		3373	85	675	Header	16.4	122.5	Header	16.4	122.5	139	139.	139	206W
Bear 11	4134	3463	.90	5.63	Header	167	2088	Header	16.4	122.5.	169	375	159	229%
Floor 10		3543	984	70.9	Header	16.4	122.5	Header	16.4	172.5	139	139	139	196%
Floor 9	4338	3629	306	726	Header	36.2	2001	Header	16.4	122.5	159	225	159	21986
Floor B		3715	511	243	Header	16.4	122.5	Header	16.4	122.5.	139	130	139	187W
Floor 7	4247	2349	1366	47.0	Header	26.5	178,5	Header	33,6	211.6	238	200	206	44378
Hoor 6		1001	-1349	200	Header	30.0	199,5	Header	75.7	170.7	201	275	201	1000%
Hoor 5	4475	-656	-41	19.7	Header	486	283.0	Header	765	1745.	773	309	223	1163%
Floor 4		1084	124	212	Header	336	2116	Header	28.3	189.3	223	241	223	102996
Floor 3	180		Ť	+		- 100					4			
Thurs ?														

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**Guy Nordenson and Associates** 

INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 62 (NORTH - SOUTH)

Floor	Column hearts	Column I mark	Column I read	Caracilo Dital		As-Desig	ASSESSMENT AND CAPACITY OF PACIFIC CORRECTIONS (ALPA)	A EVACING CORRECTIO	instration.		(Pravided by the Connections Only)	(Possided by the Connections Only)	Onnections Only	
	(Cumulative)	(Cumulative)	(Increments)	Total	Conn. Ive	North Connection T [kie]	P Ikiel	Opin Pype	South Connection 7 Ripl	P Riof	South in Tension +	South in Tension +	Minimum (Kip)	Percentage of Reg. Bracing Capacity
Read		239	239	84	Knife	8.1	84.0	Heater	838	431.5	440	162	168	3504%
Flast 47	135	208	737	4.7	Knife	103	105.0	Knife	60	978	7.3	111	7.3	1762%
Floor 46		526	48	25(	Kaife	38.1	84.0	Kaife	0.0	63.0	7.1	3	77	139296
Hoor 45	312	350	43.	029	Knife	8.1	84.0	Knite	6.0	610	71	96	7.1	1190%
Floor 44		342	43.	6.88	Knife	18	84.0	Knife	8.0	63.0	71	90	71	1039%
Elsor 43	441	3865	43	1.7	Knife	8.1	84.0	Kolfe	6.0	610	71	90	111	923%
Floor 42		426	43	8.6	Knife	8.1	840	Knife	0.0	63.0	7.1	- 86	71	63 19h
Floor 41	195	471	43	9.6	Knife	8.1	84.0	Knife	8.0	019	T.	06	77	75696
Floor 40		513	43	16.3	Knife	8.1	84.0	Knife	60	83.0	21	30	31	693%
Flast 39	693	929	43	11.1	Knife	8.1	84.0	Knife	0.9	63.6	71	- 06	71	BA098.
Hoor 35		593	43	12.0	Knite	8.1	84.0	Knife	6,0	63.0	71	90	. 1/	594%
Floor 37	818	841	43	12.8	Knife	8.1	84.0	Kinte	8.0	63.0	71	96	r.	555%
Floor 36		684	43	13.7	Knife	200	84.0	Knife.	60	623	7.1	- 80	n	520%
Fluor 35	945	726	43	14.5	Knife	8.1	84.0	Knife	6.0	83.0	21	90	311	430%
Floor 34		769	43	15.4	Knife	8.1	84.0	Knife	6.0	63.0	71	96	71.	46298
Floor 13	1071	812	43	16.2	Kaife	8.1	84.0	Knife	6.0	610	7.1	96	7.1	436%
Floor 32		858	43	121	Knife	8.1	84.0	Knife	6,0	63.0	7.1	90	.71	41-076
Huor 31	1197	898	43	18.0	Knife	8.1	84.0	Knife	6.0	610	31	90	11	390%
Floor 30		1361	43	10.8	Knife	8.1	84.0	Knife	99	63.0	7.	06	7.1	3709%
Floor 29	1323	9302	43	187	Knife	8.1	84.0	Knite	8.0	83.0	71	96	77	387%
Flaor 28		1028	43	20.6	Knife	8.1	84.0	Knife	6.0	63.6	7.1	06	11.	345%
Elnor 27	1449	101	43	21.4	Knife	8.1	84.0	Knife	6,0	63.0	71	06	71	337%
Hoor 26	-	1115	44	223	Knife	8.1	84.0	Knife	6,0	63.0	7.1	96	7.1	310%
floor 25	1575	1159	44	222	Knite	8.1	84.0	Knife	6.0	63.0	7.1	80	21	307%
Flace 24		1203	44	24.1	Knife	8.1	84.0	Knife	6.0	9729	7.1	8	71	₩.26Z
Flast 23	1701	1299	- 76	260	Knife	8.1	840	Knife	6.0.	63.0	71	90	71	-4A/2
Floor 72		1354	58	22.1	Kuife	8.1	84.0	Knife	99	830	11	96	7/	76.3%
Floor 21	1855	1409	55	200.2	Knife	1.8	84.0	Knife	09	63,0	71	06	71	252%
Floor 20		1452	44	250	Knife	8.1	94.0	Knife	0.0	63.0	71	96	71	245%
Floor 19	1995	1497	44	566	Kodle	8,1	84.0	Knife	0.0	63.0	11	06	11	230%
Hoor 18		1541	48	308	Knife	81	840	Knife	0'9	630		96		237%
Floor 17	2020	15,006	45	IN.	Knife	18	84.0	Knife	0.0	600		96	17	224%
Files In	20147	1633	40	35.6	Anne	0.0	040	Amile	000	0770	74	90	11	0.000
Dow 14	75.47	151	46.	34.4	Knife	0.0	040	Knife	0.0	000	71	96	21	20.28
Floor 13	2373	1766	45	35.3	Knife	8.1	34.0	Knife	6.0	63.0	7.1	96	77	201%
Hear 12		1011	45	36.2	Knife	8.1	84.0	Knife	6.0	63.0	71	8	11	196%
Bleer 11	2499	9581	48	37.1	Keife	8.1	84.0	Knife	60	930	.71	06	w	19268
Floor 10		1901	.48	38.0	Knife	8.1	84.0	Knife	8.0	63.0	17	- 80	71	187%
Floor 9	2625	1946	45.	389	Koffe	6.1	84.0	Knife	6.0	0'09	7.6	80	3.0	10368
Floor B		1661	45.	388	Knife	1.8	840	Knife	6.0	0.03	21	30	7.1	17996
Floor 7	2766	2002	-11	40.0	Knite	6,0	63.0	Knife	6,0	63.0	69	69	69	1,72%
Hoore		1393	-610	27.5	Trust Chic	MC	MC	Knife	10.3	105.0	WC.	NC	MC	≥-100
Hoor 5	2940	1058	-335	212	Jruss Cox	NC	MC	Knite	8.0	63.0	MC	WC.	NC.	>100
Floor 4		1130	11	228	Krille	6,0	63.0	Ruite	6.0	630	60	69	52	30508
Boor 3	1381	1181	15	578	Kulfe	200	84.0	Knife	9.0	610	28	8	22	353%
Hooe 2		1236	63	24.7	Kolfe	7.6								

design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that co

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leared	Column Lunds	Polymen fronts	Coloren Load	Carocito Dilla		As-Design	ed Axial Capacity o	As Designed Axial Capacity of Bracing Connections [Kips]	nes Kipsi			Postaded by the	(Postated by the Connections Only	
	(Cumulative)		(Increments)	Total	Conn. Type	West Connection 1 Niel	P Rice	Conn lype	East Connection 7 (Ripl	P (Rio)	West in Tension + East in Comp. [hp].	East in Tension a West in Comp (kip)	Minimum (Kip.)	Percentage of Reg. Bracing Capacity
Rost		239	239	8.5	Header	207	1448	Heater	543	2823	373	200	200	4168%
Flast 47	135	208:	-32	4.7	Header	16.4	1225	Knife	10.3	105.0	120	133	171	29229%
Floor 46		256	48	)3	Header	12.2	100.2	Kalfe	8.1	84.0	96	100	8	1882%
Hoor 45	312	350	43	029	Header	16.4	122.5	Knite	8.1	84.0	100	131	100	1000%
Floor 44		342	43.	6.8	Header	164	122.5	Knife	8.1	84.0	100	131	100	1457%
Fluor 43	441	385	43	1.7	Header	Neinfo	No linfo.	Kolle	8.1	84.0	No info	No info	No info	
Fluor 42		428	43	8.6	Header	16.4	1225.	Knife	- 81	84.0	100	101	.001	1174%
Floor 41	195	471	43	9.6	Header	No Info	No Info	Knife	8.1	84.0	Nointo	No info	Noinfo	
Floor 40		513	43	10.3	Header	16.4	122.5	Knife	1.3	840	1001	133	300	978%
Elmor 39	693	929	43	1111	Headen	No Info	No tafo	Knife	8.1	84.6	Monto	No info	No tafo	
Hoor 38		599	特	12.0	Header	16.4	122,5	Knife	8,1	0.48	100	131	100	839%
Floor 37	818	841	43	12.8	Header	16.4	122.5	Kinte	8.1	84.0	100	131	100	783%
Floor 35		584	43	13.7	Header	16.4	122.5	Knife.	123	84.0	100	131	.100	734%
Fluor 35	945	328	43	14.5	Header	164	1725.	Knife	8.1	84.0	100	131	.100	69146
Floor 34		769	43	15.4	Header	16.4	1225.	Knife	8.1	84.0	100	131	100	653%
Floor 13	1071	812	43	16.2	Header	16.4	1225-	Knife	8.1	84.0	100	131	100	61906
Floor 32		858	43	177.	Header	16.4	122.5	Knife	8.1	84.0	100	131	300	587%
Hopt 31	1197	898	43	18.0	Header	16.4	1225	Knife	8.1	84.0	100	101	100	25906
Floor 30	-	1961	43	301	Header	16.4	1725.	Knife	8.1	204.0	100	131	100	53469
Floor 29	1323	9302	43	19.7	Header	164	172.5	Knife	8.1	84.0	100	131	100	510%
Flaor 28		1028	43	20.6	Header	164	122.5	Knife	8.1	84.0	100	131	100	4884%
Elect 27	1449	1021	43	21.4	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	46996
Hoor 26	1	1115	44	22.3	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	450%
Floor 25	1575	1159	44	232	Header	16.4	1225	Knife	8.1	84.0	100	131	100	-96CE9/
Floor 24		1203	44	24.1	Header	16.4	1225-	Knife	8.1	84.0	100	131	100	416/w.
Flant 23	1701	1289	. 16	260	Header	16.4	122.5	Knife	8.1	840	100	131	100	387%
Floor 22	100	1354	-58	22.1	Header	164	1225	Knife	8.1	84.0	100	131	100	373%
Floor 21	1855	1409	55	20.2	Header	16.4	122.5-	Knife	8.1	94.0	100	(31	100	3574
Floor 20		1452	44	25.0	Header	16.4	1225	Knife	8.1	94.0	100	131	100	340%
Hoor 19	1995	1497	44	20.0	Header	16.4	177.5	Knife	8.1	340	100	131	100	31676
Hoor 18		1541	48	30.8	Header	16.4	172.5	Knife	8.1	84.0	100	131	100	3269
Flant 17.	2121	1596	45	31.7	Neader.	164	1225	Knife	8.1	84.0	100	131	100	317%
Fluor 16		1631	45	32.6	Header	16.4	1225	Kniře	8.1	84.0	100	131	100	30556
Hoor 15	2247	1676	45.	33.5	Header	16.4	1225	Knife	8.1	840	.100.	131	100	30008
Floor 14		1721	45	34.4	Header	16.4	1225	Knife	8.1	84.0	100	131	100	29296
Floor 13	2373	1766	45	35.3	Header	16.4	122.5	Knife	.8.1	840	100	131.	100	20496
Hear 12		1011	45	36.7	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	277%
Bear 11	2499	9581	45	37.1	Header	16.4	122.5.	Knife	8.1	84.0	100	131	100	2759h.
Floor 10		1061	46	380	Header	164	122.5	Knife	8.1	84.0	100	131	100	264%
Floor 9	5892	1946	46.	389	Header	16.4	1225	Knife	6.1	64.0	100	131	100	2559%
Root B		1661	46.	398	Header	164	122.5	Kaife	8.1	64.0	100	131	100	252%
Hoor 7	2766	2002	-11	40.0	Header	18.4	122.5	Knife (offset)	11.7	126.0	142	134	104	33500
Hoore		1393	-610	37.8	Header	707	1448	Knite (offset)	8.1	84.0	105	151	105	376%
Hoor 5	2940	1058	-335	212	Header	88.8	8'05%	Knile (offset)	83	84.0	174	656.	174	677%
Rince 4		1130	11	228	Header	7.9	872	Knife (offset)	8.1	840	26	Æ	98	381%
Hoor 3	1381	1181	15	208	Header	12.2	1007	Fin foffset	103.8	103.8	116	208	116	497%
Hoor 2		1235	5	24.7	Header	164	122.5	lin (offset)	103.9	103.0	120	326	120	48788

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Floor	Column brads	Design Axial Column Loads	Column I read	Canacity (29kd		As-Design	As-Designed Axial Capacity of Bracing Connections (Kips)	of Bracing Connection	nes Kipsi			(Provided by the C	foral blacing Capacity in the North-South Direction (Provided by the Connections Galo)	
	(Camulative) (Kpf	(Cumulative)	(Increments) (Kipl	Kipl	Conn. Type	North Connection T [Kip]	C (bip)	Onn. Ppc	South Connection 7 [kip]	C (Rip)	North in Tension + South in Comp. (htp)	South in Tension + North in Comp. (kip)	Minimum (Rip)	Percentage of Req. Bracing Capacity
Road		77	21	0.4	Knife	8.1	84.0	Heater	17.2	100.2	108	**	*	23384%
Floor 47	98	63	43	1.3	M-Cons	MC	- NC	Header	13.6	107.8	NC.	. WC	NC	>100
Floor 46		100	37	2.0	Knife	6,0	63.0	Header	25.0	167.0	173	250	88	4380%
Hoor 45	304	157	95	31	Knife	6,0	010	Seated	42	47	40	105	388	153306
Floor 44.		213	.86	4.3	Knife	6.0	63.0	Seated	42	42	48	105	48	1129%
Floor 43	141	. 769	96	5.4	Knife	6.0	63.0	Seated	45	45	43	105	48	819.51%
Floor 52		325	99	.65	Knife	6.0	63.0	Seared	42	45	48	105	418	739/8
Hoor 41	215	301	99	7.6	Knife	8.0	620	Seated	42	42	48	105	48	GJONA
Floor 40		438	99	86	Kalle	60	63.0	Sexted	42	42	.48	105	48	549%
State 39	713	194	.99	9.6	Knife	6.0	63.0	Sealed	42	.42	48	105	48.	ABBNik
Hoor 38		551	15	11.0	Knife	6.0	63.0	Sealed	42	47	48	105	48	436%
Floor 37	848	808	25	12.2	Knife	8.0	63.0	Scated	42	- 24	48	105	89	395%
Floor 36		665	19.	13.3	Knife	60	63.0	Seated	.45	45	438	105	48	367%
Flast 35.	586	737	25	14.4	Knife	8.0	63.0	Seated	47	42	-48	105	48	333%
Floor 34		779	27	15.6	Knife	0.0	63.0	Seated	42	45	48	105	48	306%
Floor 33	1121	837	5.8	16.7	Kaife	6.0	63.0	Seated	42	42	40	105	48	287%
Floor 32		894	58	17.9	Knife	6.0	63.0	Seated	42	42	48	105	48	269%
Floor 31	1257	952	58	19,0	Knife	6,0	63.0	Sealed .	42	45	40	105	46	252%
Floor 30		1010	- 6/8	202	Knife	6.0	63.0	Seated	42	24	48	106	46	7,00%
Floor 29	1383	1089	88	21.4	Knife	8.0	63.0	Seated	42	42	48	105	48	225%
Floor 28		1127	88	33.8	Knife	0.9	63.0	Septed	45	42	48	105	48	213%
Floor 27	6651	1106	65	23.7	Knite	6.0	63.0	Scatted	42	42	40	105	88	30209
Hoor 26		1244	629	24.9	Knife	6,0	63.0	Seated	45	45	49	105	48	193%
Floor 25	1065	1302	5.8	26.0	Knife	6.0	63.0	Scated	42	42	-48	105	48	184%
Floor 24		1361	58	27.2	Knife	90	63.0	Seated	45	42	438	105	48	176%
Fluor 23	1301	1434	E.	2007	Knife	6.0	63.0	Sealed	42	42	43	105	48	167%
Floor 22	1	1051	77	70.1	Kuife	80	670	Seated	42	47	40	108	48	1890
floor 21	1969	1351	74	31.6	Knife	60	63.0	Sexted	42	43	40	105	46	152%
Floor 20		1640	65	32.8	Knife	00	63.0	Sented	42	45	919	105	48	1469W
Hoor 19	2121	1099	53	34.0	Knife	6,0	03.0	Scaled	42	45	40	105	94	14196
Floor 18		179	09	352	Knife	6.0	63.0	Seated	45	42	49	105	89	136%
Flant 17	1522	1819	60	36.4	Knife	0.9	63.0	Seafed	42	62	AR	105	89	132%
Fluor 16		1879	09	37.6	Kuife	6.0	63.0	Seated	42	42	422	105	48	178%
Boor 15	2393	1919	09	388	Kuife	8,0	630	Sented	42	47.	48	105	48	124%
Floor 14		1989	09	40.0	Knife	09	63.0	Seated	45	45	48	105	84	120%
floor 13	5252	2029	09	41.2	Kulle	6.0	63.0	Scated	42	43	48	105	848	#411
Heyr 12	and an annual	2119	09	42.4	Knife	0'9	63.0	Scaled	24	45	2	105	8	11396
F1007 11	5992	2160	60	43.6	Knife	0,0	630	Scaled	24	26	94	60	48	11048
01 200	J. Cont.	ZZZS	3.6	447	Kaffe	900	630	Sealed	45	7.6	48	109	48	10/06
Floor 3	1097	20%		45.4	Kulle	1910	63.0	Xoned	25	76	48	10%	99	1058
FIOOR SE	2000	2337	19	47.1	Ante	600	63.0	Septent	7.	7	400	1003	96	10.5%
Tions /	7367	2418	100	930	Fine	103.0	103.0	Lin	100.0	103.0	DUC.	200	400.	4 1000
Hour C	(AFGE)	2000	3.74	900	Vertee	100.0	220	Forte	1000	105.0	244	22	5.0	1.70%
Sloce 4	2626	2752	45	550	Knile	080	630	A STATE OF THE STA	0	0	2	K3	2 19	1104
Floor 3	3545	2865	114	57.3	Knife	6.4	63.0	Kode	8.5	63.0	600	-	看	121%

C6.12

Floor	As built Axial Column brads	Column toarls	Column Iread	Canada Dilli		As-Design	red Axial Capacity of	ASTRESIDED AXIA CAJACITY OF BLACING CONNECTIONS [KIPS]	is living			(Provided by the Connections Only)	Sunections Only	
	(Cumulative)	(Cumulative)	(Increments) (Kiol	Kipl	Conn Type	West Connection T Ikini	Clispl	Onn. Prec	East Connection 7 [kip]	C Riel	West in Tension + East in Comp. [Mp]	West in Tension + East East in Tension + West in Come (No)	Minimum (Rip)	Percentage of Reg. Bracing Capacity
Read		77	21	9:0	Header	164	177.5	Header	18.4	172.5	139	139	138	3375,8%
Floor 47	98	63	43	1.3	Header	16.4	122.5	Header	. 25.0	167.0	183	148	148	11632%
Floor 46		100	37	2.0	Header	78.0	245.9	Header	17,2	126.2	205	263	205	1020398
Hoor 45	304	157	26	3.1	Header	16.4	172.5	Header	12.2	115.1	132	135	132	420096
Floor 44		213	.86	4.3	Header	164	122.5	Header	12.2	115.1	132	135	132	3097%
Floor 43	441	769	56	5.4	Header	16.4	122.5	Header	12.2	115.1	132	195	132	2447%
Floor 52		325	99	6.6	Header	16.4	1225	Header	12.2	116.1	132	196	102	2024%
Floor 41	215	3111	95	7.6	Header	164	122.5	Header	12.2	115.1	132	196	1302	1725%
Floor 40		438	96	88	Header	16.4	122.5	Header	12.2	115.1	137	135	.132.	150316
Fluor 39	713	494	.99	9.6	Header	16.4	1225.	Header	12.2	115.1	132	135	137	13319h
Hoor 38		551	15	11.0	Header	16.4	122.5	Header	12.2	115.1	132	135	132	119496
Floor 37	849	808	25	12.2	Heater	16.4	122.5	Header	12.2	115.1	192	195	112	106290
Floor 36		989	25	13.3	Header	16.4	122.5	Header	12.2	115.1	132	106	132	9894
Flaor 35.	982	727	57	14.4	Header	164	122.5	Header	12.2	115.1	132	135	132	911%
Floor 34		179	57	15.6	Header	16.4	122.5	Header	12.2	115.1	132	135	132	04400
Floor 13	1121	837	5.8	16.7	Header	16.4	122.5-	Header	19.2	115.1	117	135	132	785%
Floor 32		894	85	17.9	Header	16.4	122.5	Header	12.2	115.1	132	132	132	735%
Floor 31	1257	952	29	19.0	Header	16.4	122.5	Header	12.2	1351	132	135	132	96069
Floor 30	-	1010	- 878	202	Header	16.4	172.5	Header	17.7	11511	132	136	110	65198
Floor 29	1393	1089	88	21.4	Header	164	177.5	Header	17.2	115.1	137	136	137	61596
Floor 28		1127	85	22.5	Header	164	122.5	Header	12.2	115.1	132	135	(32	583%
Floor 27	1529	1106	63	23.7	Header	16.4	122.5.	Header	12.2	115.1	132	135	132	555%
Hoor 26		1244	63	24.9	Header	16.4	1225	Header	12.2	115.1	132	135	132	529%
Floor 25	1665	1302	58	26.0	Header	16.6	122.5	Header	12.2	115.1	132	135	132	505%
Floor 24		1361	58	27.2	Header	16.4	122.5	Header	122	135.1	132	135	132	485%
Fluor 23	1901	1434	73	2007	Header	16.4	1225-	Header	16.4	137.3	164	139	139	434%
Floor 22		1051	73	30.1	Header	16.4	1225	Header	16.4	137.1	154	129	139	4619a
Floor 21	1969	1051	74	31.6	leader	16.4	132.5	leader	164	137.3	154	138	139	43900
Floor 20		1640	65	32.8	Header	16.4	122.5	Header	12.2	115.1	132	105	132	4010%
Hoor 19	2171	1099	53	34.0	Header	16.4	177.5	Heady	12.2	118.1	132	135	137	387%
Floor 18		1769	09	352	Header	18.6	172.5.	Heater	12.2	115.1	132	135	110	374%
Flant 17.	1522	1819	60	364	Header	164	122.5	Header	12.2	115.1	132	135	132	36296
Fluor 16		1879	0.9	37.6	Header	16.4	1225	Header	12.2	115.1	137	135	132	350%
floor 15	2393	1939	09	388	Header	16.4	1225	Header	12.2	115.1	132	135	132	33906
Floor 14		1989	09	40.0	Header	16.4	1225	Header	12.2	118.1	132	135	132	32996
Floor 13	5253	2059	09	41.2	Header	16.4	122.5	Header	12.2	115.1	132	135	132	319%
Heyr 12		2119	09	42.4	Header	16.4	122.5	Header	12.2	115/1	192	135	132	310%
Floor 11	2665	2180	60	43.6	Header	16.4	122.5.	Header	12.2	118.1	137	135	182	30%%
Floor 10		2235	98	447	Header	164	172.5	Header	12.2	11511	132	135	132	294%
Floor 9	2001	2296	6.1	45.9	Header	16.4	1225	Header	12.2	115.1	132	135	132	289%
Floor 8		2323	61	47.1	Header	16.4	122.5.	Header	12.2	115.1	132	135	132	279W
Floor 7	2962	2478	121	49.6	Header	16.4	129.9	Header	16.4	122.5	139	146	139	280%
Hoore		2532	CS.	50.6	Header	17.6	8701	Header	12.2	100.2	118	156	118	96712
Hoor 5	3242	2703	1/1	54.1	Header	898	3508	Header	393	241.3	331	368	331	613%
Hoor 4		2752	69	980	Header	12.2	100.2	Header	16.4	122.5	135	111	111	212%
Floor 3	3545	2865	114	57.3	Header	35.0	262	Header	70.7	1447	180	20%	180	314%
Floor 2		1606	2.3	202	,									

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64 (NORTH -	
- COLUMN	
CATALOGUE	
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LUMN BRACI	
INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 64 (NORTH - SOUTH)	
C6.13	

Floor	As built Axial Column brads	Design Axial Column Loads	Column I read	Capacity (296)		As-Design	ed Axial Capacity	As-Designed Axial Capacity of Bracing Connections [Kips]	ns [Kips]			Provided by the Connections Only!	(Provided by the Connections Only)	
	(Cumulative)	(Cumulative)	(Increments)			North Connection			South Connection		North in Tension +	South in Tension +	Minimum (bin)	Percentage of Req.
9 0	lkpl	Kipl	[Kip]	(Kip)	Conn. Type	T Displ	C Ripl	Conn. Type	Thip	Clopl	South in Comp. (ktp)	Worth in Comp. [kip]	Died mannen	Bracing Capacity
Dian 27	187	254	104	5.1			0	Kofe	140	2300	230	14	14	27.70
Floor 46		385	135	2.0		00	0	Knife	6,0	63.0	63	. 9	9	7796
Hoor 45	537	472	84	976		0	0	Knife	6.0	63.0	63	9	9	Palok
Floor 44		555	83	11.1	X	9	0	Knife	6.0	63.0	63	19	8	E-10k
Floor 43	722	638	83	12.8	1	0	ō	Knife	6,0	63.0	63	.9	.9	47%
Floor 52		719	82	14.4		. 0	.0.	Knife	6.0	63.0	63	9	9	4776
Floor 41	906	891	82	16.0		0	0	Knife	8.0	63.0	139	9	8	17.0
Floor 40		883	18	17.7		0	0	Knife	60	63.0	63	9	9	34%
Floor 39	1090	- 345	82	19.3		.0	0	Knife	09	63.0	E3	9	14	3346
Floor 38		1046	10.	20.9		0	0	Kinte	6,0	63.0	13	9	io	9,675
Floor 37	1274	1128	82	22.8		.0	.0	Kinte	8.0	63.0	63	w	8	27%
Floor 36		1210	K7	242		0	ō	Knife	40	63.0	.63	4	4	2696
Fluor 35	1458	1551	82	258	X	0	0	Knife	6.0	63.0	63	. 8	ig.	25%
Floor 34		1374	82	27.5		0	.0.	Knife	6.0	63.0	63	9	.9.	220h
Floor 33	1642	1456	82	29.1		0	0	Knife	0.0	610	63	9	9	7196
Floor 32		1538	82	36.8	5	0	0	Knife	6.0	63.0	63	.9	9	20%
Floor 31	1826	1620	20	32.4		.0	0	Knife	0'9	63.0	63	9	9	1996
Floor 30		1702	239	340		- 0	0	Knife	0.9	63.0	63	20	ië.	trens
Floor 29	2010	1785	83	35.7	ľ	. 0	.0	Knife	8.0	83,0	63	. 9	- 180	17%
Floor 28		1868	82	37.4	7	.0	0	Knife	6.0	63.0	63	9	(6)	10%
Floor 27	2194	1950	0.3	39.0		0	0	Knife	6.0	63.0	E9	9	io.	1508
Hoor 26		2033	0.3	40.7	4	0	.0.	Knife	6,0	63.0	63	8	. 9	1801
Floor 25	2378	2116	#3	.423		0	6	Knife	6.0	63.0	63	9	9	-14m
Floor 24		2199	83	44.0	*	0	0	Knife	6.0	63.0	63	16	16	1488
Fluor 23	2992	2303	104	46.1		.0	0	Knife	6.0	63.0	63	.9	9	1300
Floor 22		2407	104	40.1		0	-0	Knife	6.0	63.0	63	19	te	9671
Floor 21	2332	2512	105	502	1	0	.0	Knife	6.0	63.0	63		9	125
Floor 20		2597	90	519		0	0	Knife	0.0	63.0	63	9	9	1546
Hoor 19	2999	2602	92	53.6		0	0	Knife	6,0	63.0	63	9	9	1106
Floor 18		2766	90	553		0	0	Knife	6.0	63.0	19	9	9	1300
Floor 17.	3183	2851	RS	629		0	0	Knife	6.0	63.0	63	g	9	1995
Fluor 16		2936	818	58.7		0	0	Knife	6.0	63.0	63	9	9	1000
Floor 15	3367	3021	. 90	80.4		0	.0	Knife	6.0	63.0	13	19	ie.	ton
Floor 14		3106	0.5	62.1		. 0	.0	Knife	6.0	63.0	63	14	9	1001
Floor 13	3551	-3192	90	63.8		0	ō	Knife	6.0	63.0	63	ış.	9	88
Heyr 12		3278	602	939		.0	ō	Knife	0.9	63.0	63	.9	9	19.65
Floor 11	3735	3363	90	67.3		. 0/	0	Knife	6.0	63.0	63	19	io	9661
Floor 10		3450	87	69.0		0	0	Knife	6.0	63.0	6.1	to.		1861
Floor 9	3919	3507	10.7	707	,	0	.0	Knife	6.0	63.0	63	œ	9	BW
Floor 8		3623	.90	72.5	ě.	0	0	Knife	6.0	0.09	E9	.9	16	Brill
Hoor 7	3754	3306	317	66.1	Knife	6.0	63.0	Truss Cnx	NC	NC	NC	NC.	NC	>100
Roor 6		3306	0	66.1	X	0	.0	Fin	96.5	86.5	87	87	- 87	43196
Hoor 5	4120	3219	-68	64.4	e d	0	ō	Truss Clor.	NC	NC	NC	NC.	MC	>100
Floor 4		3371	153	67.4		0	0	Ŧ	86.5	88.5	-87	8/	87	128%
Floor 3	4568				5	0	0	File	272.8	2768	£			*
Thomas ?														

Floor	As built Axial	Design Axial Column Locals	Design Axial	Reg Coun Axial		As-Design	ned Axial Capacity o	As-Designed Axial Capacity of Bracing Connections (Kips)	ns [Kips]			Total Brading Capacity in the East-West Uirection (Posseded by the Connections Color	the East-West Direction Sometime Only	
	Cumulative		Burrement	The state of the s		West Connection			Eact Connection		West in Tention a Fact	Eact in Tention a West	The state of the s	Deposit and Bed
	Repl		IKipl	(Kip)	Conn. Spec	T Dipl	C (66)	Conn lype	Thip	Citiol	m Comp. [htp]	in Comp (kip)	Minimum (Kip)	Bracing Capacity
Road		184	184	3.7	Huss, Onc.	TUC:	NC	Header	582	298.1	M	M	NC.	>100
Flux 47	157	254	3/0	5.1	Header	43.5	241.3	Header	283	1883	233	271	.233	4587%
Floor 46		389	50.	7.0	Header	187	7697	Header	77.1	289.5	339	346	339	436400
Hoor 45	537	472	-04	9.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1470%
Floor 44		698	28	1111	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1252%
Fluor 43	11.1	638	17.3	128	Header	16.4	1725	Header	164	122.5	139	139	139	1085%
Fluor 42		719	62	14.5	Header	16.4	1225.	Header	16.4	122.5	139	139	139	9699
Floor 41	906	80.1	23)	16.0	Header	164	122.5	Header	164	122.5	139	1339	139	3167%
Floor 40		8883	141	17.7	Header	16.4	1225	Header	16.4	122.5	139	130	139	787%
Elistic 39	1090	985	11.2	193	Header	16.4	1225.	Header	16.4	122.5.	139	139	139	720%
Hoor 35		1046	U)	20.9	Header	16.4	1225	Header	16.4	122.5	139	139	139	66496
Floor 37	1274	1128	2.9	228	Header	18.4	122.5	Header	18.4	122.5	158	159	139	84649
Flaor 36		1210	17.7	292	Heades	164	122.5	Header	164	122.5	139	139	139	574%
Fluor 35	1458	1257	82	258	Header	164	1225.	Header	16.4	1225	139	139	139	538%
Floor 34	H	1374	0.5	205	Header	16.4	1225.	Header	16.4	122.5	139	136	139	90009
Floor 13	1642	1456	62	29.1	Header	16.4	1225	Header	No Info	No Info	No info	Noinfo	Notato	,
Floor 32		1538	11.2	36,8	Header	16.4	122.5	Header	Neinfo	Notate	Normo	Nomfo	Nombo	
Hupt 31	9281	1620	0.5	324	Header	16.4	1225	Header	Notinfo	No folio	No info	No info	No info	
Floor 30		1702	(0)	340	Header	16.4	1725	Header	Notinfo	Notes	No info	No info.	No info	
Floor 29	2010.	1785	83	34.7	Header	164	1725	Header	16.4	177.5	139	139	139	389%
Floor 28		1868	82	37.4	Header	164	1225	Header	16.4	1525	139	138	639	3726
Eleoc 22	2194	1950	0.3	39.0	Header	16.4	122.5.	Header	16.4	122.5	139	139	139	356%
Hoor 26		2033	63	40.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	34296
Floor 25	2378	2116	H3	423	Header	16.4	122.5	Header	164	122.5	139	139	139	328%
Flace 24		2199	83	44.0	Header	16.4	1225	Header	16.4	1225	139	139	139	310=
Hust 23	2562	2303	104	46.7	Headet	16.4	1225.	Header	20.7	1443	161	143	143	31108
Floor 22		2407	104	46.1	Header	164	1225	Header	20.7	1443	161	141	143	297%
Floor 21	2392	2512	105	503	Header	16.4	1225-	Header	164	122.5	138	(38)	139	277%
Floor 20		2597	90	519	Header	16.4	122.5	Header	16.4	1225	139	139	130	260%
Hoor 19	2999	2602	98	53.0	Header	16.4	1725.	Header	16.4	172.5	139	139	139	259%
Hoor 18		2766	90	553	Header	16.4	172.5.	Header	364	172.5	139	139	139	257%
Floor 17:	3183	2851	N.S.	62.0	Neader.	164	1225	Header	16.4	122.5	139	138	139	244%
Fluor 16		29.66	312	536.7	Header	16.4	1225	Header	16.4	5221	139	139	139	237%
Hoor 15	3367	3021	. 50	50.4	Header	16.4	1225	Header	16.4	122.5	139	138	139.	23086
Floor 14		3010	100	62.1	Header	16.4	1225	Header	16.4	122.5	139	139	139	224%
Floor 13:	3551	-3193	96	63.0	Reader	16.4	122.5	Header	16.4	122.5	139	108	139	21996
11(s)r 12		3278	612	656	Header	16.4	122.5-	Header	16.4	122.5	139	139	139	312%
Bear 11	3735	1363	98	673	Header	16.4	1225.	Header	16.4	122.5	139	139	139	707m
Floor 10		3450	RZ	69.0	Header	164	122.5	Header	164	172.5	119	139	139	201%
Floor 9	3919	3237	117	707	Header	16.4	1225	Header	16.4	122.5	139	139	139	1968
Floor B		3623	116.	72.5	Header	16.4	1225.	Header	164	1225	139	130	139	1924
Hoor 7	3754	3306	-317	64.1	Header	135.7	4388	Header	1943	227.3	J.	MC	MC	>100
Hoore		3306	0	199	Header	707	1448	Truns Chx	NC.	NC	WC.	NC	JN.	>100 >100
Hoor 5	4120	3219	-418	544	Header	26.5	174.5	Truss Clus	JE.	MC	MC	WC	MC	>100
Roce 4		3371	183	674	Header	28.3	1893	huso Cox	.au	MC	MC	MC.	MC	> 100
Boor 3	4568			+					- 10	1	4	- 4		
Those 2		1			1	-			Y	1			1	

Notes 1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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Floor	As built Axial Column brads.	Design Axial Column franks	Column I carl	Reg Coon Axial Canacity Eyliki		As-Desig	ned Axial Capacity s	As-Designed Axial Capacity of Bracing Connections [Kips]	ans [Kips]			lotal Stacing Capacity in the North-South Direction (Provided by the Connections Cally)	eing Capacity in the North-South Direction Provided by the Connections Only	
	(Camulative)	(Cumulative)	(Increments)	West	one fee	Morth Connection	logi o	Other Liber	South Connection	P. Palat	North in Tension +	South in Tension +	Minimum (Ep)	Percentage of Reg.
Bool	Model	213	213	4.3	Koife	10.7	1050	Knife Knife	12.4	1280	TOR TORE	117	-Ot	976am
Flast 47	764	787	150	-875	Knife	14.0	2100	Knife	128	189.0	203	223	503	3420%
Floor 46		317	20	6.3	Knife	0'9	63.0	Kalfe	0.9	63.0	69	69	69	106566
1001.45	411	338	21	60	Knife	6,0	61.0	Knite	6,0	610	69	69	69	1021%
Floor 44		356	21	12	Knife	6.0	63.0	Knife	6.0	63.0	609	69	69	962%
Fluor 43	464	380	-21	7.5	Knife	6,0	63.0	Kolfe	6,0	63.0	69	669	69	900%
Fluor 42		401	21	60	Knife	6.0	63.0	Knife	. 09	610	69	69	89	805 196
Floor 41	115	421	.21	8.4	Knife	8.0	63.0	Knife	8.0	019	69	100	699	81596
Kor 40		442	21	8.8	Kolfe	- 60	63.0	Knife	60	83.0	69	69	69	781%
luor 39	135	463	31	83	Knife	0.0	913	Knife	09	63.6	8	639	69	7.469W.
Floor 38		483	23	1.6	Knite	6.0	63.0	Knife	6,0	63.0	69	69	6.9	21496
Floor 37	603	504	21	101	Knife	8.0	63.0	Kinte	8.0	63.0	69	28	28	584%
Floor 36		629	50	10.5	Knife	60	63.0	Knife.	6.0	0.130	69	49	69	H295W
Flaor 35	649	545	21	10.9	Knife	8.0	63.0	Knife	6.0	63.0	69	69	69	63.3%
loor 34		596	21	113	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	61006
Floor 13	1989	547	-21	11.7	Kaife	6.0	63.0	Knife	6.0	610	69	69	639	588%
Floor 32		. 203	.21	12.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	60	569%
Hupt 31	741	829	21	12.6	Knife	6,0	63.0	Knife	6.0	630	63	69	69	24006
Floor 30		669	21	13.0	Knife	0.9	630	Knife	0.9	63.0	69	60	69	63208
Hoor 29	787	6889	21	124	Knife	-80	63.0	Knife	80	83.0	69	59	689	5.15/10
Floor 28		089	21	13.8	Knife	6.0	63.0	Knife	6.0	63.6	-89	66	- 69	500%
less 22	833	111	21	14.2	Knife	6,0	63.0	Knife	6.0	63.0	69	69	98	485/86
Hoor 26		731	.21	14.6	Knife	6,0	63.0	Knife	6,0	63.0	69	69	89	47759
Floor 25	879	155	21	350	Knife	0.9	63.0	Knife	6.0	63.0	69	69	69	455%
Fluor 24		773	17	15.5	Knife	0.9	63.0	Knife	6.0	63.0	69	69	69	####
Fluor 23	925	789.	36	16.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	432%
Floor 22		824	26	16.5	Knife	8.0	6.10	Knife	. 0'9	610	639	28	69	419%
Floor 21	5863	850	36	17.6	Knife	60	63.0	Knife	6.0	63.0	63	69	69	406%
Floor 20		871	.21	17.4	Kulle	60	63.0	Knife	60	63.0	69	69	69	390%
Floor 19	1035	893	7.1	17.9	Kolle	6,0	03.0	Knife	6.0	63.0	69	63	69	387%
Floor 18		914	.21	10.3	Knife	6.0	63.0	Knife	6,0	63.0	69	89	69	378%
Flant 17	1081	935	21	18.7	Knite	60	019	Knife	6.0	83.0	68	69	69	369%
Fluor 16		956	21	19.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	36746
loor 15	1127	878	21	361	Knife	8.0	630	Knife	6.0	63.0	58	59	69	35366
300r 14		666	21	20.0	Knife	6.0	63.0	Knife	6.0	620	69	. 69	69	34598
Floor 13	1173	1001	21	20.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	3389
Nor 12		1042	.21	20.8	Knife	6,0	63.0	Knife	0.9	63.0	69	63	69	33196
Hear 11	1219	1084	11	2113	Kolfe	6,0	63.0	Knife	6.0	630	69	69	609	324%
Floor 10		1096	7.7	21.7	Knife	60	63.0	Knife	80	630	69	69	69	31,0%
Floor 9	1265	1107.	22	22.1	Kalle	6.0	63.0	Knife	6.0	009	69	69	69	3129
Floor B		1129	22	22.5	Knife	6.0	0.03	Knife	6.0	000	69	69	69	300W
Floor 7	1336	1166	37	223	Truss Clox	MC	NC	Truss Croc	MC	NC	NC	NC.	MC	>(00)
Hoore		1197	31	23.9	Fin	865	86.5	Fin	121.1	121.1	- 208	208	208	B67m
Hoor 5	1416	1012	-185	202	Truss Cux	NC	MC	Truss Cinc	NC.	MC	MC	WC	MC	>100
Floor 6		1031	2	20.6	Į.	88.5	88.8	Ē	1211	127.1	508	208	208	1000%
Floor 3	1539	1578.	547	31.6	Trues, Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Troop 2		9091	28	02.1	Uh	121.1	12121	Ties	200.4	207.6	326	322	722	100190

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Hoor	Column triads	Column Loarls	Column Ired	Capacity (2%)		As-Design	ned Axial Capacity	As Designed Axial Capacity of Bracing Connections [Kips]	ans [Kips]			Provided by the Connections Only)	universions Only)	3
	(Cumulative)	(Oumulative)	(Increments)	1	20.00	West Connection	1 100 00		East Connection	0.000	West in Tension + East	East in Tension + West	Minimum (Rip)	Perce
0.0	ldoj	Idiy	(Kip)	(dv)	Conn lype	1000	C (bp)	Conn lype	(100)	Club	in Comp. Dipl	in Comp. (kip)	2002	20
POST CO.	-	28.2	213		Nume	200	1980	neader.	683	1837	407	197	187	1
Han 41	704	787	88	9.8	Header	16.4	1225	Header	875	233.8	750	1683	741	-
Floor 46		245	20	6.3	Header	16.4	1225	Header	202	144/8	161	143	143	
Hoor 45	411	338.	21	09	Header	16.4	1225	Header	16.4	122.5	139	138	139	
Floor 44		359	21	7.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	
Elsor 43	464	380	21	7.5	Header	16.4	1725	Header	164	122.5	139	139	139	
Floor 42		401	21	- 80	Header	16.4	1225	Header	16.4	122.5	139	139	139	
Floor 41	115	421	.21	8.4	Header	164	172.5	Header	16.4	122.5	139	139	139	
Floor 40		442	21	8.8	Header	16.4	1725	Header	16.4	122.5	139	1311	139	
Fluor 39	. 257	463	21	9.3	Header	16.4	1225.	Header	16.4	122.5.	139	139	139	
Hoor 35		483	17	6.7	Header	16.4	122,5	Header	16.4	122,5	139	139	139	
Floor 37	603	504	21	101	Header	18.4	122.5	Header	18.4	122.5	139	139	139	
Floor 36		629	50	10.5	Header	164	122.5	Header	164	172.5	139	139	- 139-	
Fluor 35	649	546	23	10.9	Header	16.4	1225.	Header	Nome	No lefu	No info	We info	No info	
Floor 34		566	21	113	Header	16.4	1225.	Header	No-Info	No Info	Noinfo	Me info	No info	
Floor 13	1982	547	21	11.7	Header	16.4	1225-	Beader	No Info	No info	No info	No info	No info	
Floor 32		.203	.21	12.1	Header	164	122.5	Header	Ne Info	So lafe	Nomo	No mo-	No rafo.	
Fluor 31	741	929	21	126	Header	16.4	1225	Header	16.4	1225	139	139	139	Н
Floor 30		609	21	13.0	Header	16.4	1725.	Header	16.4	1725	139	133	139	
Floor 29	787	688	21	124	Header	164	1725	Header	16.4	172.5	139	-109	139	
Floor 28		690	21	13.8	Header	16.4	122.5	Header	16.4	122.5	138	138	639	Ц
Eleoc 22	833	711	23	14.2	. Header	16.4	122.5.	Header	16.4	122.5	139	139	139	Ц
Hoor 26		731	.21	14.6	Header	16.4	1225	Header	16.4	122.5	139	139	139	
Floor 25	878	155	21	350	Header	16.4	122.5	Header	16.4	122.5	139	139	139	Н
Flace 24		773	21	15.5	Header	16.4	122.5	Header	16.4	1225.	139	139	139	
Fluid 23	928	789	26	16.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	4
Floor 22		824	26	16.5	Header	16.4	1725	Header	16.4	177.5	139	139	139	
Floor 21	5863	920	36	17.0	Header	16.4	122.5	Header	16.4	122.5	138	(38)	139	
Floor 20		871	21	17.4	Header	16.4	122.5	Header	16.4	122.5	139	139	130	4
Hoor 19	1035	893	71	17.9	Header	16.4	1725	Header	16,4	1725	139	139	139	Ц
Floor 18		914	7.1	10.3	Header	16.6	172.5	Heater	36.6	172.5	139	139	139	
Flant 17	1981	935	21	18.7	Neader.	16.4	122.5	Header	16.4	122.5	139	139	139	
Fluor 16		956	21	19.1	Header	16.4	5221	Header	16.4	127.5	139	139	139	4
Hoor 15	1127	878	21	361	Header	16.4	1225	Header	16.4	122.5	133	139	139	
Floor 14		866	21	20.0	Header	16.4	1225	Header	IEA .	122.5	138	139	139	
Floor 13	1173	1021	21	36.4	Header	16.4	122.5	Header	16.4	122.5	139.	139	139	Н
Hear 12		1042	.21	20.8	Header	16.4	122.5-	Header	16.4	122.5.	139	139	(39	Ц
Hear 11	1219	1064	22	2113	Header	16.4	172.5.	Header	16.4	122.5	139	139	139	Ш
Floor 10		1096	22	21.7	Header	16.4	122.5	Header	164	172.5	119	139	139	
Floor 9	1265	1107.	22	22.1	Header	16.4	1225.	Header	16.4	122.5	139	100	139	Ц
Floor B		1129	22	22.5	Header	164	1225.	Header	16.4	1225.	139	130	139	_
Hote 7	1336	1166	37	223	Header	16.4	122.5	Header	12.2	1002	117	135	117	
Hoore		1197	31	23.9	Header	16.4	122.5	Header	17.9	129.9	146	140	140	
Hoor 5	1416	1012	-185	202	Header	16.4	1225	Header	42.2	256.1	273	165	165	
Floor 4		1001	11	20.6	Header	16.4	177.5	Header	12.7	1002	117	135	111	4
Boor 3	1539	1578.	(44)	37.6	Header	76.5	1745-	Header	13.6	211.00	738	208	208	
Hoor 2		909	28	02.1	Header	16.4	1225	Header	75.0	167.0	183	140	148	

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WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

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ned based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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**Guy Nordenson and Associates** 

Countichter  Cou	Hoor Ask	As built Axial Column brads	Design Axial Column Loads	Column frail	Canacity (24)		As-Desig	As-Designed Axial Capacity of Bracing Connections (Kips)	of Bracing Connectic	ne [Kips]			form of a control of the Connections Calvi	lotal Bracing Capacity in the North-South Direction (Provided by the Connections Caly)	
Figor   Figo		Jamulative)	(Cumulative)	(Increments)			North Connection		1	South Connection		North in Tension +	South in Tension +	Moinsum (Bot	Percentage of Req.
126   126   127   127   127   127   128		lkipi	Kipl	[Kip]	[Kip]	Conn. Type	Tikipl	C Ripl	Conn. Type	Thip	Clippl	South in Comp. [ktp]	Worth in Comp. [kip]	Defect management	Bracing Capacity
956         456         1172         9.1         0           756         460         1171         9.1         0         0           756         460         1171         9.1         0         0         0           756         160         11.11         0         0         0         0         0           160         165         160         11.11         0<	100	126	1364	99	37	)	0	0	Korfe	14.0	2300	210	14	- 74	382%
9.66         4465         111         9.3         0         0           784         563         103         1134         0         0         0           1802         756         103         1134         0         0         0         0           1803         1866         103         1131         0	94		356	172	7.1		0	0	Knife	0'9	63.0	63	.9	9	865B
784         653         113         114         0         0           776         165         1151         0         0         0           1107         264         116         151         0         0         0           1107         847         1161         1         0         0         0           1270         1162         1170         1182         0         0         0           1270         1162         1171         0         0         0         0           1270         1162         1170         1182         0         0         0         0           1186         1190         116         27.24         0         0         0         0         0           1186         1190         111         27.24         0         0         0         0         0           1186         1187         27.24         0 <td>45</td> <td>395</td> <td>466</td> <td>111</td> <td>93</td> <td></td> <td>0</td> <td>0</td> <td>Knite</td> <td>10.3</td> <td>105.0</td> <td>105</td> <td>10</td> <td>10</td> <td>110%</td>	45	395	466	111	93		0	0	Knite	10.3	105.0	105	10	10	110%
784         953         105         1131         0         0           11022         286         100         151         0         0         0           1720         1192         100         210         0         0         0           1720         1193         100         2710         0         0         0           1720         1193         100         2731         0         0         0           1746         1482         91         2734         0         0         0           1746         1482         91         2734         0         0         0           1746         1482         91         2734         0         0         0           1746         1482         91         2734         0         0         0           1746         1482         91         2734         0         0         0         0           1746         1482         91         2724         0         0         0         0         0         0         0         0         0         0         0         0         0         0         0         0         0	44		588	103	11.4	X	9	.0	Knife	6.0	63.0	63	9	8	53%
1700   1766   1102   115.1   1.0   0   0   0   0   0   0   0   0   0	43	784	853	115	13.1	-	0	0	Koife	6,0	63.0	63	. 9	.9	461
1007   945   940   1453   5   0   0   0   0   0   0   0   0   0	15		756	102	15.1		0	.0	Knife	9.0	63.0	63	.9	9	ACON
1720   1947   1942   1823	41	1002	845	06	16.9		0	0	Knife	8.0	610	63	. 9	S	TERM
1720   17050   1701   2710	40		947	102	18.9		0	0	Knife	60	63.0	63	9	9	1600
1488   1789   1792   25.24   0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	39	1226	1050	163	210	A	- 0	0	Knife	0.9	63.0	63	.9	9	5000
1488   1789   1416   275.4   0   0   0   0   1445	38		1153	103	23.1		0	0	Knife	6,0	63.0	29	19	10	26%
1360   811   277.7   0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	37	1438	1269	116	25.4	,	. 0	0	Septed	42	42	. #2	42	42	185%
1446   1445   912   7826   9   9   9   9   9   9   9   9   9	35		1380	93	27.7		0	0	Knife	40	63.0	.63	9	. 46	377%
1544   941   30.9   0   0   0   0   0   0   0   0   0	35	1746	1452	255	29.0	X	.0	0	Knife	0'9	63.0	63	.8	18	76.00
1938   1635   392   32.7   - 0 0 0 0     27150   1022   395   34.8   - 0 0 0 0     27150   1022   395   34.8   - 0 0 0 0     27150   1022   395   34.8   - 0 0 0     27150   1022   395   34.8   - 0 0     27151   27162   392   42.1   - 0 0     27152   27162   392   42.1   - 0 0     27152   27162   392   42.1   - 0 0     27152   27162   393   42.1   - 0 0     27152   27162   393   42.1   - 0 0     27153   27162   31.1   42.1   - 0 0     27154   27162   31.1   42.1   - 0 0     27155   27162   31.1   42.1   - 0 0     27156   27162   31.1   42.1   - 0 0     27156   27162   31.1   42.1   - 0 0     27156   27162   31.1   42.1   - 0 0     27156   27162   31.1   42.1   - 0 0     27156   27162   31.1   - 0 0     27156   27162   31.1   - 0 0     27156   27162   31.1   - 0 0     27156   27162   31.1   - 0 0     27156   27162   31.1   - 0 0     27156   27162   31.1   - 0 0     27156   27162   31.1   - 0 0     27156   27156   31.1   - 0 0     27156   27156   31.1   - 0 0     27156   27157   31.1   - 0 0     27156   27157   31.1   - 0 0     27156   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   27157   31.1   - 0 0     27157   31.1	34		1543	16	30.9	1	0	0	Knife	.09	63.0	63	9	.9.	1908
1727   1727   1845   0   0   0   0   0   0   0   0   0	13	1938	1635	- 26	32.7	ı	0	0	Knife	6.0	0.09	69	ję.	9	1994
2150   1002   955   9844   9   9   9   9   9   9   9   9   9	32		1771	92	34.5	5	0	0	Knife	0'9	63.0	63	.6	9	NAC1
78872         71016         816         3784         0         0           7514         2180         812         4421         0         0         0           7514         2180         812         4421         0         0         0         0           7784         2180         917         4458         0	31	2150	1822	- 36	36.4		.0	0	Knife	6.0	63.0	63	9	.9	10%
7857 7001 815 407 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	30	-	1918	365	30.4	V	. 0.	0	Knife	17.6	109.0	169	13	180	T. Tribi
2514         27163         842         4211         0         0           7709         27288         942         4438         0         0         0           7709         27308         942         448         0         0         0         0           7709         27302         113         4148         0         0         0         0           7709         27603         113         4458         0         0         0         0           7709         27604         111         4458         0         0         0         0           7708         27604         111         4458         0         0         0         0           7709         27604         111         4458         0         0         0         0         0           7713         27606         141         4458         0         0         0         0         0           7754         3706         145         640         0         0         0         0         0         0         0           7778         375         375         275         0         0         0         0         <	58	2822	2011	88	402		0	.0	Knite	8.0	63.0	83	9	39	1698
7514         72166         91         41.9         -         0         0           7709         7200         91         45.8         -         0         0         0           7709         7200         91         47.6         -         0         0         0           7709         7200         117         49.5         -         0         0         0           7709         7700         117         49.5         -         0         0         0           7700         116         54.2         -         0         0         0         0           7700         116         54.2         -         0         0         0         0           7700         117         54.2         -         0	28		2303	8.5	42.1	,	0	.0	Knife	6.0	63.0	63	19		14%
2706         72788         197         44.88         -         0         0           2706         7207         117         44.88         -         0         0           2708         2807         117         44.85         -         0         0           2709         118         44.42         -         0         0         0           3138         2806         117         56.5         -         0         0           315         2806         117         56.5         -         0         0           315         2806         117         56.5         -         0         0           315         2806         117         56.5         -         0         0           315         3106         117         56.5         -         0         0         0           315         3108         186         184.1         -         0	27	2514	2196	93	43.9	ı	0	0	Knite	6.0	63.0	19	ia	ia	1406
7706 77312 813 415 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	26		2288	26	45.8	3	0	0	Knife	6,0	63.0	63	B	d	130
2586         7445         91         485         -         0         0           7709         115         54.8         -         0         0           7709         116         54.2         -         0         0           7704         116         54.2         -         0         0           7706         117         56.5         -         0         0           7707         2000         94         56.4         0         0         0           7704         7015         95         60.2         0         0         0         0           7754         7706         96         66.0         0	52	2706	2382	9.3	47.6		0	9	Knife	6.0	63.0	63	9	9	13%
2886         7592         117         51.8         0         0           7138         7705         118         54.2         0         0           7138         7705         117         54.2         0         0           7754         7705         94         56.5         0         0           7754         7705         94         56.4         0         0           7754         7705         94         56.4         0         0           7778         7706         94         66.7         0         0           7778         7706         94         67.7         0         0           7778         7707         94         67.7         0         0           7778         7707         97         77.5         0         0           4114         7677         97         77.5         0         0         0           4408         773         74         0         0         0         0           4408         752         77.2         0         0         0           4408         75.2         77.2         0         0         0	54		2475	93	49.5	1	0	0	Knife	6.0	63.0	63	19	16	12%
7709   1116   54.2   0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	52	2898	2692	117	518	1	.0	0	Knife	6.0	63.0	63	.9	lió.	1200
7138 7106 117 5645 - 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	7.7		2709	118	54.2		0	0	Knife	9.0	63.0	63	39	ia)	1794
1754   7500   94   5614   0   0   0     1754   7307   95   6023   0   0   0     1754   7307   95   6622   0   0   0     1755   7307   96   6611   0   0   0     1756   7307   96   6611   0   0   0     1757   7307   96   7117   0   0   0     1757   7307   96   7117   0   0   0     1757   7307   96   7756   0   0   0     1757   962   962   7756   0   0   0     1757   963   965   7724   0   0   0     1757   9640   965   972   0   0     1757   9641   772   7644   0   0     1757   9641   772   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9641   772   0   0     1757   9642   772   0   0     1757   9642	2	3138	2826	117	56.5	1	00	.0	Kulfe	09	63.0	63	.9	(g)	1946
1754   2015   595   6023	20		2020	z	584		0	ō	Knife	0.9	63.0	63	9	9	1000
Ti546         1710         95         547.2         0         0           1738         1380         96         6840         0         0           1738         1380         96         6840         0         0           2738         1366         96         6839         0         0           2930         2461         96         673         0         0           4472         3478         96         71,7         0         0         0           4474         3972         94         77.5         0         0         0           4468         1967         406         77.3         604         0         0           4468         1968         177         0         0         0         0           4468         1967         406         77.3         604         0         0           4468         1968         1772         0         0         0           4488         1967         157         0         0         0           4488         1968         173         0         0         0           4488         1967         157         0 <td>19</td> <td>3354</td> <td>3015</td> <td>96</td> <td>603</td> <td>· ·</td> <td>0</td> <td>0</td> <td>Knife</td> <td>0'9</td> <td>63.0</td> <td>63</td> <td>9</td> <td>9</td> <td>1004</td>	19	3354	3015	96	603	· ·	0	0	Knife	0'9	63.0	63	9	9	1004
1756   1756   186   1861   -	18	200	3110	36	622		0	0	Knife	0'9	63.0	139	4	9	1794
3738 37300 195 6840 - 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	17	3546	3205	*	64.1		0	0	Knife	60	63.0	63	2	4	3.00
31738   31584   3169   3169   31738   31584   3169   3169   3173   3173   3173   3173   3173   3173   3174   317	91		3300	96	66.0	j.	0	0	Knife	6.0	63.0	63	9	9	340
293-06         248-11         185-         68-88         9-9         0	91	3738	3336	96	679		0	0	Knife	0'9	029	3	00	100	366
79590 78587 896 7117 - 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	*		1491	48	68.8	1	0	0	Knife	0'9	63.0	16	la	9	940
4172 3178 56 77.56 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	0	3830	3507	96	71.7		0	ō	Knife	6.0	63.0	63	100	9	
4112 3178 96 775	12		3602	96	73.6		0	ō	Knife	09	63.0	63	9	9	98
4314 5107 1976 70 10 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	=	4122	3778	36	75.6		0	.0	Knife	6.0	63.0	63	9	io.	969
4314 3872 817 784 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	10		3875	35	27.5		0	0	Knife	6.0	63.0	63	19	· S	6590
4088 3689 166 2114 20 6 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	6	4314	3972	.28	79.4		0	.0	Knife	6.0	63.0	. 63	9	æ	8.8
4088 2664 -404 72.3 Kulfe 6.0 63.0 63.0 4400 3.69 -5.5 772 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	-8		4068	.96	814	0	0	0	Kaife	0.9	0.09	63	.6	tá	742
4489 3450 1722 0 0 0 0 6481 3850 185 7729 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	1	4088	3664	-404	733	Knife	6.0	63.0	Truss Crox	NC	NC	NC	NC	NC	>100
6468 3459 757 688 6441 0 0 0 5 637 6481 0 0 0 6 7 6 6 7 6 6 7 6 6 7 6 6 7 6 6 7 6 6 7	9		3610	13.	72.2	X	0	0	Fin	121.1	121.1	121	121	121	168%
729 0 0 0	5	4488	3458	.152	269	1	0	ō	Truss Crox	NC	NC	NC	MC	NC	>100
			3643	185	72.9	1	0	0	F	988	88.5	87	87	87	119%
	é	4943									+	4			. 4.

Theory AHI
News 1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

Hoor As built Axial	Axial Design Axial	Column I cord	Picq Loan Axian		AS-INSI	As Designed Axia Capacity of Macing Connections (April	OF ENALTHY COUNTY	furbul su			(Possided by the Consections Dalid	(Postated by the Consections Dollar	
		(Increments)	and the section		West Connection			East Connection		West in Tension + East	East in	The state of the s	Percentage of Red
lich		(Kip)	(Kip)	Onn lype	T Dipl	C Bipl	Conn Type	7 (Ripl	C (kip)	in Comp. [htp]	in Comp (kip)	Minimum (Rp)	Bracing Capacity
Rost		118	7.4	Header	47.7	7561	Header	207	1462	187	-277	187	795710
Flax 47 126		- 99	3.7	Heades	283	1883	Header	707	144.8	174	210	174	47367k
Floor 46	980	721	7.1	Header	78.0	795.0	Header	68.0	322.9	705	383	383	5,382%
Hoor 45 566		111	6.0	Header	16.4	122.5	Header	29.3	189.1	206	152	152	1620%
	609	103	11.4	Header	16.4	122.5	Header	20.7	144.8	161	143	.143	1259%
Fluor 43 784		115	13.1	Header	16.4	172.5	Header	25.0	0791	101	140	149	117.9%
Floor 42	756	102	151	Header	16.4	1225.	Header	20.7	144.6	191	143	143	940%
Floor 41 1002		06	16.9	Header	164	1925	Header	25.0	167.0	1001	140	148	8720
		102	18.9	Header	16.4	1225	Header	20.7	1408	181	143	143	75610
6 lister 39 1220	0501 0	163	250	Header	16.4	1225.	Header	25.0	167.0	143	146	148	70296
Hoor 38	1183	103	23.1	Header	16.4	122.5	Header	20.7	144/3	161	1/13	143	62106
Floor 37 1438		116	25.4	Header	16.4	122.5	Header	79.7	469.7	420	202	202	797%
Floor 36	1380	-81	27.7	Header	164	1225	Header	164	122.5	138	1.19	139	511%
Fluor 35 1746	H 1452	7.65	280	Header	16.4	1225.	Header	30.0	5031	197	153	163	529W
Floor 34	1543	16	30.9	Header	16.4	1225.	Header .	16.4	122.5	139	109	139.	450%
Floor 13 1938	1615	-26	32.7	Header	No Info	No info	Header	30.0	1919	Noinfo	Noinfo	No info-	
Floor 32	1771	92.	34.5	Header	No info	No lafe	Header	16.4	122.5	Neinfo	No to Fo	No info	
Hupt 31 2150		96	36.4	Header	No Info	No lafo	Header	362	20603	No tefo.	No info	No info	
Floor 30	1918	-96	301.4	Header	No info	No info	Header	707	1948	Noinfo	thouto.	No info	
Floor 29 7572	7 7011	933	402	Header	16.4	172.5	Header	30.0	189.9	187	153	153	379%
Fluor 28	2103	642	42.1	Header	164	1225	Header	16.4	123.5	138	139	139	330%
Eleor 27 2514		93	43.9	Header	16.4	122.5.	Header	30.0	1809	197	153	153	347%
Hoor 26	2288	. 265	45.8	Header	16.4	1225	Header	164	122.5	139	139	139.	3040%
flor 25 2706	2382	93	47.6	Header	16.4	1225	Header	36.0	180.9	161	151	153	320%
	12	93	49.5	Header	16.4	1225	Header	16.4	122.5	139	129	139	287%
Huse 23 2898	24	112.	518	Header	707	1443	Header	36.2	2063	230	181	181	34996
		116	542	Header	20.7	14438	Header	20.7	14471	165	185	165	3050%
Floor 21 3138		117	585	leader	16.4	122.5-	Header	36.2	2080	375	689	159	201%
-		94	50.4	Header	16.4	122.5	Header	164	122.5	135	100	130	238%
Hoor 19 3354		90	60.3	Header	16.4	172.5	Header	30.0	1009	161	163	153	253%
		96	623	Header	16.4	172.5	Heater	164	172.5	139	139	139	223%
Flant 17 3546		98	54.1	Neader	164	1225.	Header	30.0	180 B	197	183	153	238%
	79	382	66.0	Header	16.4	1225	Header	16.4	127.5	138	138	139	.2114h
Hoor 15 3738		.90	613	Header	16.4	1225	Header	30.0	6 0xd	107	15,1	153	225%
-		98	69.8	Header	16.4	1225	Header	16.4	1325	139	554	139	19996
Floor 13 3930		28	71.7	Neader	16.4	122.5	lkader	300	100.9	197	153	153	21396
4	1	98	73.6	Header	16.4	122.5	Header	16.4	122.5	139	(39	(3)	189%
Hear 11 4122		.96	75.6	Header	16.4	1225.	Header	30.0	6081	161	153	153	702%
		38	311	Header	16.4	122.5	Header	16.4	172.5	139	1,159	139	17996
Floor 9 4314		.76	784	Header	16.4	1225.	Header	36.0	1889	161	151	153	19298
	7	.96	FX8	Header	16.4	1225.	Heater	16.4	1225	NC.	MC	NC	>100
Hoor 7 4088		-404	733	Header PI	184.1	316.3	Header PI	147.7	399,0	NC.	NC	MC	>100
		-22	77.2	Trust Ow	NC.	24	Truss Cho.	NC	NC	MC	NC	JW.	>100
Hoor 5 4488		.152	2.69	Tress Clar	NE	MC	Truss Clas	ME	MC	MC	WC.	MC	>100
Rice 4	3643	185	729	Junes Care	MC	MC	Truss Cox	No.	NC	MC	. WC.	MC	2,100
Floor 2					-					4			

Notes 1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

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Floor	Column hands	Column Locals	Column I cod	Carperite Dilla		As-Desig	As Designed Axial Capacity of Bracing Connections [Kips]	of Bracing Connectio.	ns Kips]			(Possibled by the Connections Only)	Powerfeet by the Connections United	
	(Camulative)	(Camulative)	(Increments)	The state of the s		Morth Connection			South Connection		North in Tension +	South in Tension +	1	Percentage of Red
	lkpl	lkipl	lKipl	(Kip)	Conn Type	1 1566	C lispl	Conn Type	7 Big	C (kip)	South in Comp. (htp)	North in Comp. (tup)	Minimum (Ep)	Bracing Capacity
Rost		191	141	2.8	Knife	6.0	630	Knife	80	830	699	689	69	2445%
Flast 47	11	238	.78	4.8	Knife	14.0	210.0	Knife	12.4	128.0	140	227	146	2945W
Floor 46		303	59	6.1	Knife	6,0	63.0	Kalfe	0'9	63.0	63	63	69	113946
1007.45	390	365	62	7.3	Knife	6,0	63.0	Knite	6,0	610	83	83	8	940%
Floor 44		434	69	83	Knife	8.0	63.0	Kuffe	8.0	63.0	69	69	69	834%
Fluor 43	513	465	- 21	176	Knife	60	63.0	Kolfe	0.0	63.0	69	669	69	743%
Flavor 42		540	69	10.3	Knife	6.0	63.0	Knife	0.0	610	69	88	8	672%
Floor 41	675	564	15	11.1	Knife	8.0	63.0	Knife	8.0	013	69	69	69	81296
Floor 40		612	49	12.2	Knife	60	63.0	Knife	60	83.0	69	69	69	564%
Elmor 39	737	663	20	13.3	Knife	6.0	63.0	Knife	. 09	63.0	69	69	69	5219k
Hoor 38		711	49	14.2	Knite	6,0	63.0	Knife	6,0	63.0	69	63	656	48554
Floor 37	849	174	9	16.5	Scated	42	24	Kinte	8.0	63.6	105	84	89	310%
Floor 36		816	42	16.3	Knife	.09	63.0	Knife.	600	029	69	49	89	423%
Fluor 35	988	380	43	17.7	Knife	8.0	63.0	Knife	6.0	63.0	69	69	69	401%
Floor 34		905	42	180	Knife	6.0	63.0	Knife	6.0	63.0	69	69	639	38396
Floor 13	1050	946	44	18.9	Kaife	6.0	63.0	Knife	6.0	610	69	69	63	36506
Floor 32		906	42	19.8	Knife	6.0	63.0	Knife	6.0	63.0	69	60	69	349%
Hupt 31	1174	1039	21	20.8	Knife	6,0	63.0	Knife	0'9	63.0	69	60	69	33296
Floor 30		1094	68	219	Knife	17.6	1090	Knife	17.6	1090	202	202	202	92198
Floor 29	1404	1126	31	325	Knife	8.0	63.0	Knite	8.0	83.0	639	539	609	307%
Flaor 28		1156	30	23.1	Knife	6.0	63.0	Knife	80	63.0	-89	69	95	299%
Elsor 22	1472	1188	32	218	Knife	6.0	63.0	Knife	6.0	63.0	69	699	699	29106
Hoor 26		1218	30	24.4	Knife	6,0	63.0	Knife	6,0	63.0	69	69	89	283%
floor 25	1580	1249	35-	75.0	Knife	6.0	63.0	Knife	6.0	63.0	639	69	69	275%
Fluor 24		1280	36	25.6	Knife	6.0	63.0	Knife	6.0	9729	69	69	69	270%
Hays 23	1608	1319	39	26.4	Knife	6.0	63.0	Knife	6.0	63.0	- 69	69	63	262%
Floor 22		1356	0	27.1	Kuffe	6,0	62.0	Knife	9.0	63.0	69	28	69	254%
Floor 21	1692	1395	36	27.9	Kulfe	60	63.0	Knife	0.0	63.0	678	69	89	24746
Floor 20		1425	30	20.5	Kvile	00	63.0	Knife	6.0	63.0	69	69	8	242%
Hoor 19	1768	1457	32	79.1	Kolle	6,0	63.0	Knife	0.0	63.0	69	69	69	237%
Floor 18		1488	10	798	Knife	6.0	63.0	Knife	6,0	63.0	9	8	69	232%
Floor 17.	1836	1520	-22	36.4	Keife	60	63.0	Knife	6.0	83.0	69	92	69	227%
Fluor 16		1881	31	3170	Knife	6.0	63.0	Knife	6.0	63.0	639	69	69	22300
Hoor 15	1904	1583	35	51.7	Knife	0'9	630	Knife	0.9	62.0	58	69	69	216m
Floor 14		1614	31	323	Knife	60	63.0	Knife	6.0	62.0	49	69	69	214%
floor 13:	1972	1646	31	32.9	Knife	6.0	63.0	Knife	6.0	970	69	69	69	210%
Nor 12		1291	31	33.5	Knife	6.0	63.0	Knife	09	63.0	63	69	69	20678
Blear 11	2070	1710	33	342	Koife	6.0	63.0	Knife	60	63.0	69	69	639	2020s
Floor 10		1741	15	34.8	Knife	60	630	Knife	8.0	63.0	69	69	69	196%
Floor 9	2100	1774	33	35.5	Koffe	6.0	63.0	Knife	6.0	63.0	69	69	69	194%
Floor B		1006	31	36.1	Knite	6.0	.010	Knife	6.0	.0703	69	69	69	191%
Floor 7	2201	1782	-23	38.6	Truss Chx	NC	NC	Truss Cho.	MC	NC	NC.	MC	MC	>100
Hoore		1824	19	365	Hir	865	86.5	Fin	121.1	1717	- 208	208	208	569%
Hoor 5	2328	1425	-389	28.5	Tress Cux	NC	NC.	Truss Cinc	MC	NC	W.	WC	MC	3-100
Roce 4		1466	1	787	File	988	888	Œ.	121.1	127.1	208	208	208	123%
Floor 3	2423	1958	225	385	Trues, Chix	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Thank D		1907	200	200	True.	0.00	1000		A 40.00 W	0.000	4000	-		1000000

1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

I I	As built Asiai	Design Axial	Contract land	The Party		As-Desig	As Designed Axial Capacity of Bracing Connections [Kips]	Firsting Connects	ons [Kips]			Country Capacity of the Edward Country of the Count	local bracing capitally in the East-west Ulrection	
961	(Camulative)	(Cumulative)	(Increments)	rapaga (Car		West Connection			East Connection		East	East in Tension a West	Morman (tro)	Percentage of Req.
	lkpl	(Kipl	lkipl	(Kp)	Conn. Type	T Dipl	c lippl	Conn Type	7 (hip)	Cikipl	in Comp. [htp]	in Comp. (kip)	Districtions (Nobel	Bracing Capacity
Roof		191	141	7.8	Header	16.6	1725	Header	184	172.5	139	1339	138	497,6%
Flax 47	II	238	.78	48	Header	.750	1670	Header	7.07	1448	170	188	1/0	3571%
Floor 46		303	59	6.1	Header	202	144.9	Header	20.7	8***	591	591	.165	2731%
Hoor 45	390	365	62	7.3	Header	16.4	122.5	Header	293	189.3	206	152	152	2090%
Floor 44		414	49	8.3	Header	164	122.5	Header	164	122.5	139	139	139	1675%
Elsor 43	513	465	51	973	Header	16.4	1725	Header	207	14431	161	143	143	1547%
Floor 42		513	-69	103	Header	16.4	1225	Heater	16.4	1225	139	139	139	1354%
Floor 41	675	564	15	11.1	Header	164	1925	Header	7.02	1448	181	143	143	1270%
Floor 40		612	49	12.2	Header	16.4	1225	Header	16.4	122.5	139	130	139	113540
Elizor 39	737	563	20	13.3	Header	16.4	1225.	Header	20.7	144.5	191	143	143	1081%
Hoor 38		711	49	14.2	Header	16.4	1225	Header	16.4	1225	139	139	139	9/6/6
Floor 37	848	1734	63	16.5	Header	16.4	122.5	Header	-62.2	256.1	273	165	185	10658
Floor 36		816	42	16.3	Header	164	127.5	Heater	164	122.5	138	1389	139	-915.1%
Flaor 35	98%	350	43	17.2	Header	No tafo	No lafo.	Header	30.0	150.5	Noinfo	Mainfo	No info	
Floor 34		905	42	180	Header	No Info	No Info	Header	16.4	122.5	Noinfo	No info	No info	
Floor 33	10501	946	44	18.9	Header	No Info	No Info	Header	30.0	1909	Noinfo	Noinfo	No info	
Floor 32		906	42	19.8	Header	Noinfo	No lafe	Header	16.4	1225	Meinfo	No info	No infe	
Huor 31	1174	1039	51	20.8	Header	16.4	122.5	Header	29.3	1893	202	757	751	331%
Floor 30		1094	95	21.9	Header	16.4	172.5	Header	12.0	256.1	273	165	165	38298
Floor 29	1404	1178	31	325	Header	164	122.5	Header	0.08	180.9	197	153	153	677%
fluor 28		1156	30	23.1	Header	164	1225	Header	16.4	5021	138	139	638	601%
Elsor 27	1472	1188	32	218	Header	16.4	122.5.	Header	30.0	180.9	161	153	153	64296
Hoor 26		1918	30	24.4	Header	16.4	1225	Header	16.4	122.5	139	139	139	570%
Floor 25	1540	1249	32-	75.0	Header	16.4	1225	Header	39.0	180.9	161	151	153	810%
Flace 24	1	1299	36	25.6	Header	16.4	1225	Header	16.4	122.5	139	139	139	243m
Flant 23	1608	1319	39	26.4	Header	16.4	1225.	Header	30.0	180.9	167	163	153	578%
Floor 22		1356	n	27.1	Header	16.4	1225	Header	164	177.5	139	139	139	512%
Floor 21	1692	1395	.19	27.9	fleader	16.4	122.5	Header	0.00	6001	167	153	153	54748
Floor 20		1425	30	265	Header	16.4	122.5	Header	164	122.5	139	100	139	487%
Hoor 19	1761	1457	33	79.1	Header	16.4	177.5.	Header	30.0	150.9	161	163	153	523%
Hoor 18		1488	31	79.8	Header	16.4	172.5	Heater	16.4	172.5	139	139	139	46746
Floor 17.	1836	1520	-22	30.4	Header.	164	1225	Header	30.0	185 B	197	153	153	502%
Fluor 16		1881	33	3370	Header	16.4	5221	Header	16.4	127.5	139	138	139	440%
Hoor 15	1904	1583	35	51.7	Header	18.4	1225	Header	30.0	6081	107	151	153	482%
Roor 14		1614	31	32.3	Header	12.2	1002	Header	12.2	100.2	112	112	112	348%
Floor 13:	1972	1646	31	32.9	Header	16.4	122.5	Header	300	162.4	178	153	153	46299
11(c)r 12		1291	31	33.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	414%
Bear 11	2070	1719	33	34.2	Header	16.4	122.5.	Header	30.0	1433	160	163	153	4467m
Floor 10		1741	31	348	Header	16.4	122.5	Header	16.4	172.5	139	1339	139	399%
Floor 9	2108	1774	33	35.5	Header	16.4	1225.	Header	36.0	162.4	179	153	153	430%
Floor B		9001	31	36.1	Header	16.4	122.5.	Header	16.4	1225	139	130	139	385/4
Floor 7	2201	1782	-23	38.6	Header	12.2	1002	Header	20,7	1443	157	121	121	339%
Hoore		1824	19	36.5	Header	17.9	129.9	Header	17.9	129.9	148	148	148	40504
Hoor 5	2328	1425	-356	28.5	Header	42.2	256.1	Header	42.2	756.1	268	2662	298	1047%
Hoce 4		14%	=	28.7	Header	12.7	100.2	Header	17.7	1002	112	112	112	397%
Floor 3	247.1	1958	225	38.5	Header	33.6	211.6	Header	13.6	211.6	245	245	245	629%
Winner		NO.	200	246.2	Hander	36.70	1620	Han-lie	0.36	10.20	100	1000	1000	2000

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Floor	As built Axial Column brads	Design Axtal Column Loarls	Column I read	Req Conn Axial Capacity (24%)		As-Design	ed Axial Capacity	As-Designed Axial Capacity of Bracing Connections [Kips]	ons [Kips]			lotal Bracing Capacity in the North-South Direction (Provided by the Connections Only)	he North-South Direction Connections Only)	
	(Camulative)	(Cumulative)	(Increments)		Contract and	North Connection	- Const	4	South Connection	4.00	North in Tension +	South in Tension +	Minimum (Np)	Percentage of Req.
Read	ldol	1d 24	(Mp)	0.8	Knife Knife	80	63.0	CORR. INDE	i (Kip)	0 0	South in Comp. Dripl	North in Comp. (tup) 63	*	Shacing Capacity
Flax 47	-15	1/8	47	- 73	Knife	12.4	126.0	)	0	0	12	126	12	739%
Floor 46		136.	52	2.7	Knife	6,0	63.0		0	0	9	63	9	32299
Hoor 45	346	208	72	4.2	Knife	6.0	63.0		.0	0	9	63	9	14598
Floor 44		275	19	55	Knife	6.0	63.0	X	0	ò	9	63	de la	10906
Floor 43	501	331	95	99	Knife	60	63.0	,	0	0	. 9	63	.9	70.16
Floor 42		398	29	0.0	Knife	60	63.0		0	0	9	63	9	7696
Floor 41	988	458	0.9	9.2	Knife	8.0	63.0		0	0		63	is:	Printer.
Floor 40		525	67	10.5	Knife	60	63.0	,	0	0	9	63	9	865
Floor 39	808	593	68	119	Knife	6.0	63.0		.0	0	4	63	9	5108
Hoor 38	10000	099	89	13.2	Knife	090	63.0		0	0	to a	23	io I	MENA
Floor 37	362	97/	/0	2 6	Knife	80	630			0 0	0 4	2 0	¢ 0	400
Flace 35	1117	867	67	17.3	Knife	000	630	6	0	0	- 12	20 29	é le	HOM:
Floor 34		931	89	18.6	Knife	0.9	63.0		0	0	. 9	69	.9.	3200
Floor 33	1271	1000	69	20.0	Knife	80	63.0	,	0	0	29	139	st.	966
Floor 32		1068	69	21.4	Knife	6.0	63.0	3	0	0	19	63	9	79m
Hoor 31	1425	1141	73	22.8	Knife	6,0	63.0		0	0	9	63	9	26%
Floor 30		1216	78	243	Knife	17.6	1090		0	0	13	1009	13	Ruds
Floor 29	1847	12/8	47	756	Knife	80	636		0 0	0 0	E 4	29	at G	mak?
Floor 22	1785	1402	12	28.0	Knife	60	63.0		0	0	45	2 29	ia	23.048
Floor 26		1463	29	293	Knife	60	63.0	3	0	0	9	63	9	2104
Floor 25	1923	1525	29	30.5	Knife	6.0	63.0		0	0	9	63	9	20,000
Floor 24		1587	79	31.7	Knife	99	63.0	x	0	0	15	9	ف	1594
Floor 23	2061	1684	78	33.3	Knife	6.0	63.0		.0	0	9	63	.9	186e
Floor 22		1742	78	34.8	Knife	8.0	8.1.0	X	0	0	9	63	te	121
Floor 21	7231	1920	78	36.4	Knife	.09	63.0	,	0	0	9	63	٠	174
Floor 20	19486	1882	29	37.6	Knife	000	63.0		0 0	0 0	15 25	2 5	d ti	16%
Hone 18	X300	2002	20	40.1	Knife	900	630			0	0 12	19	0 16	Tron
Floor 17	2523	2009	- 69	41.4	Knife	60	63.0	,	0	0	ts	29		15%
Floor 16		. 2131	29	42.6	Knife	6.0	63.0	x	0	0	9	63	9	1400
Floor 15	2661	2194	63	43.9	Knife	0'9	63.0		. 0	0	.9	63	19	1407
Floor 14		2257	13	45.1	Knife	6.0	63.0	X	0	0	. 9	63	9	136
Floor 13	2799	2321	63	46.4	Knife	6.0	63.0		0	ó	9	63	9	19%
Heyr 12		2384	63	47.7	Knife	0'9	63.0	X	0	0	9	63	٥	139h
Floor 11	2937	2447	63	403	Knife	60	630		0	0	20	63	ia.	120%
Floor 10	- Annah	TSI	110	205	Knife	80	630		0	0 0	= <	29	2	17.4
Floor 9	5707	4/67	173	636	Knife	000	0.3					2 0	£	1.100
Hoor 7	3223	2452	186	49.0	Trum Dux	NC	NC	Knife	8.0	63.0	NC	NC.	NC	>100
Floor 6		7574	177	51.5	Hn	121.1	121.1	,	0	0	121	121	121	235%
Floor 5	3542	2127	-446	42.5	Truss Clux	NC	MC		0	0	NC	NC	MC.	>100
Floor 4		2028	-88	40.6	Ē	121.1	121.1	x	0	o	121	121	121	299%
Floor 3	3831	2677	648	53.5	Truss Onc	NC	NC		.0	0	NC	NC	NC	>100
0.000														

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Floor	As built Axial Column hunds.	Design Axial Polymer Locals	Colomo Load	Reg Coun Axial Canacito Dillai		As-Design	ed Axial Capacity o	As-Designed Axial Caparity of Bracing Connections (Kips)	ns [Kips]			Total Brading Capacity in the East-West Direction (Provided by the Connections Cally)	the East-West Direction onnections Only	
	(Camulative)	(Cumulative)	Uncrements	(Kind	Conn. Ivor	West Connection 1 Ikiel	C Biol	Cohn Pype	East Connection (Third	C (kiel	West in Tension + East in Como (No)	East in Tension a West	Minimum (Kip)	Percentage of Reg Bracing Capacity
Roaf	Name of the last	45	73	0.00	Header	164	172.5	Header	16.4	172.5	139	139	139	184709
Flast 47	15	284	42-	173	Header	25.0	1670	Header	7.07	1448	170	188	170	10138%
Floor 46		136-	25	2.7	Header	283	1883	Header	33.6	2116	. 241	223	223	821796
Hoor 45	346	208	77	4.2	Header	164	122.5	Header	20.7	1440	191	143	143	344796
Floor 44		275	19	5.5	Header	164	122.5	Header	164	122.5	139	139	139	2526%
Fluor 43	501	331	95	6.6	Header	16.4	1725	Header	No Info	No liste	Motinfo	Mointo	No info	1
Flave 42		398	67	8.0	Header	16.4	1225.	Header	Nethfo	No lafo	Writifo	No info	Mo info	
Hoor 41	959	458	0.9	26.	Header	164	1225	Header	No info	No Info	Nointo	Nointo	Noinfo	
Floor 40		525	67	10.5	Header	16.4	1225	Header	No tafo	Notato	Meinfe	No info	No info	
Fluor 39	508	593	89	11.9	Header	16.4	1225.	Header	16.4	1225	No info.	No info.	No info	
Hoor 38		680	89	13.2	Header	16.4	1225	Header	16.4	122.5	139	139	139	1052%
Floor 37	5963	728	29	14.8	Header	16.4	1225	Header	18.4	722.5	664	128	139	954%
100 3E		795	67	15.9	Header	164	122.5	Header	164	122.5	139	108	139	87,7%
Fluor 35	1117	983	- 67	17.3	Header	164	1225.	Header	164	1225	139	139	139	80508
Floor 34		931	68	18.6	Header	16.4	1225.	Header	16.4	122.5	139	139	139	746%
Floor 33	1221	1000	69	20.0	Header	16.4	1225	Header	16.4	122.5	139	139	139	699/W
Floor 32		1068	69	23.4	Header	16.4	122.5	Header	16.4	1225	138	139	139	8200
Hupt 31	1425	1141	73	22.6	Header	16.4	1225.	Header	16.4	172.5	139	139	139	18609F
Floor 30		1216	75	243	Header	16.4	1725.	Header	13.6	2116	228	198	156	18457B
30r 29	1847	1278	67	75.8	Header	164	1725	Header	16.4	177.5	139	-139	139	S44m
Flast 28		1340	62	26.8	Header	164	1225	Header	16.4	122.5	138	139	673	519%
Elmr 27	1785	1402	- 62	28.0	Header	16.4	122.5.	Header	16.4	122.5	139	139	139	496%
Hoor 26		1463	62	293	Header	16.4	122.5	Header	16.4	122.5	139	139	139.	475%
floor 25	1923	1525	-29	30.5	Header	16.4	1225	Header	16.4	122.5	139	1339	139	456%
Flace 24		1587	29	31.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	438m
Huse 23	2061	1684	78	33.3	Header	16.4	1225.	Header	16.4	122.5	139	1339	139	4178
Floor 22		1742	7.8	34.8	Header	164	1225	Header	16.4	1775	139	138	139	399%
Roor 21	2231	1820	78	364	lleader	16.4	122.5-	Header	164	122.5	138	(0)	139	382%
Floor 20		1802	62	37.6	Header	164	122.5	Header	16.4	122.5	135	139	130	369%
Hoor 19	2305	1944	29	30.9	Header	16.4	1725.	Header	16,4	1725	139	139	139	357%
Floor 18		2002	. 29	40.1	Header	16.6	172.5	Header	38.6	172.5	139	139	139	346%
Floor 17.	2523	2009	67	41.4	Header.	164	1225.	Header	16.4	122.5	139	139	139	33696
Fluor 16		2131	29	42.6	Header	16.4	1225	Header	16,4	127.5	139	139	139	326%
Hoor 15	2661	2194	63	43.9	Header	18.4	1225	Header	16.4	1225	136	138	136	34786
Floor 14		2257	121	45.1	Header	12.2	1002	Header	12.2	100.2	112	112	112	24986
Floor 13	2799	2321	63.	46.4	Header	16.4	122.5	Header	16.4	122.5	.109.	108	139	288%
Hior 12		7354	63	47.7	Header	16.4	122.5-	Header	16.4	122.5	139	139	(3)	291%
Bior 11	2937	2447	63	40.0	Header	16.4	1225.	Header	16.4	122.5	139	139	139	789%
Floor 10		2511	64	505	Header	164	122.5	Header	164	172.5	119	139	139	27756
Floor 9	3075	2574	64	515	Header	16.4	1225	Header	16.4	122.5	139	109	139	270%
Floor B		2638	64	57.8	Header	164	1225.	Header	16.4	1225.	139	138	139	26396
Floor 7	3223	2452	106	49.0	Header	72.1	194,9	Header	72.1	6,861	NC	NC.	NC	>100
Hoore		2574	177	515	Truss Chic	NC.	NC	Truss Chx	MC	NC	MC.	NC	NC	>100
Hoor 5	3542	2227	446	42.5	Truss Cjur	Æ	NC.	Truss Clor	ME	MC	MC	WC.	MC	>100
Roce 4	100	2028	2	40.6	Jues Cux		MC	Intro Core	W	DK.	MC	26	W	>100
Hoor 3	3831	26//	55-2	878	Truss Cax	J.	NC.	Truss Cmx	MC	No.	W.	MC	MC	0000
2 2001		37.6	272	650	Teass One	NC.	MC	Truss Cox	NC.	- NAC	27-60 574 85.00 Tous-Cuz NC NC Tous-Cux NC NC NC NC	W.	MC	>100

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Hoor													
level	Column brads	Column toarls	Column Iread	Capacity (29%)		As-Design	ned Axial Capacity	As-Designed Axial Capacity of Bracing Connections [Kips]	ns [Kips]			Total Bracing Capacity in the North-South Infection (Provided by the Connections Only)	onnection
	(Camulative)	(Cumulative)	(Increments)		1	North Connection		1000	South Connection		North in Tension +	South in Tension +	Minimum (kin)
	licol	Kipl	[db]	[Kip]	Conn. Type	T [kip]	C Ropl	Conn. Type	T Displ	Clapl	South in Comp. [kip]	North in Comp. (kip)	
Read		280	980	1.8		0	0	Knite	8.0	83.0	23	8	
Float 47	126	124	44	.52		- 0	0	Knife	10.3	105.0	105	30	
Floor 46		357	233	7.1		0	0	Knife	12.4	126.0	126	12	
Hoor 45	530	499	142	10.0	-	0	0	Knife	12.4	126.0	126	12	
Floor 44		627	128	12.5	X	0	0	Knife	12.4	126.0	128	12	
Floor 43	799	670	43	13.4	1	0	-0	Kulfe	12.4	126.0	126	12	
Floor 62		813	144	163		0	.0	Knife	12.4	126.0	126	12	
Floor 41	1901	863	69	17.3		0	0	Knife	12.4	126.0	17.6	12	
Floor 40		1014	151	20.3		0	.0	Knife	12.4	126.0	126	12	
Floor 39	1335	1134	120	22.7	A	.0	0	Knife	12.4	126.0	971	12	
Floor 38		1255	121	25.1		0	0	Knife	12.4	126.0	126	12	
Floor 37	1603	1388	133	37.8	,	0	.0	Kinte	86	147.0	147	10	
Floor 36		1494	108	28.8		0	ō	Knife	103	105.0	105	.10	
Floor 35	1977	1091	107	32.0	X	.0	-0	Knife	10.3	0.501	105	10	
Floor 34		1707	106	34.1		. 0	.0.	Knife	10.3	105.0	105	- 10	
Floor 33	2219	1815	107	363	1	.0	0	Knife	10.1	105.0	105	10	
Floor 32		1921	107	38.4	3	0	0	Knife	10.3	105.0	105	10	
Floor 31	7461	2031	110	40.6		.0	0	Knife	10,3	1050	105	10	
Floor 30		2141	110	42.8		. 0	0	Knife	10,3	105.0	105	10	
Floor 29	2703	7249	109	45.0	X	. 0	.0	Knife	10.3	105.0	105	10	
Floor 28		2358	108	47.2	,	0	0	Knife	10.3	105.0	105	10	
Floor 27	2945	2467	109	49.3		.0	.0	Knife	10.3	105.0	105	10	
Hoor 26		2577	110	51.5	4	0	.0	Knife	10.3	105.0	105	10	
Floor 25	3187	2607	110	53.7		0	9	Knife	10.3	105.0	105	10	
Floor 24	0.00	2795	108	653	x	0	0	Knife	10.3	1050	105	10	
Fluor 23	3429	2941	145	58.8		.0	0	Knife	8.1	84.0	84	8	

1 The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection 2695

4697

4896

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

		Column house	Column Iran	Canarille Diffel		AS-Design	As-Designed Axial Capacity of Bracing Connections [Kips]	F Bracing commects	ins [Kips]			Printed By The Compections Could	COMPACTIONS CLIMAT	
0.4	(Cumulative)	(Cumulative)	(Increments)	Kipl	Conn. Proc.	West Connection T [Kip]	C (Sp)	Conn. Pape	East Connection 7 [kip]	C (kip)	West in Tension + East in Comp. [Hip]	East in Tension a West in Comp (kip)	Mammum (Rip)	Percentage of Req. Bracing Capacity
PKR20		280	80	18	Header	20.7	144.8	Header	28.0	167.0	158	170	170	106229%
Floor 47	126	124	44	.25	Header	7.07.	144.8	Header	707	144.8	165	165	165.	6879%
Floor 46		250	233	7,1	Header	080	122.9	Header	61,0	2007	308	365	385	539146
1007.45	530	456	291	10.0	Header	707	1443	Header	29.3	189.3	210	1/4	1/4	174510
Floor 44	100	957	128	12.5	Header	707	144.8	Header	29.3	1897	210	1/4	1/4	1385%
F100F43	199	6/0	24	13.4	Header	250	167.0	Header	Metallo	No INTO	MC	JW.	NC.	>100%
Floor 52	1	813	144	16.3	Header	707	194.6	Header	583	1881	210	174	6/1	1070%
15 tool 41	1901	5987	68	177	Header	25.0	187.0	Header	Mo Info	No Info		W	MC	\$100%
Hoor 40		1014	181	703	Header	707	144/2	Header	583	1893	210	1/4	1/4	805910
Histor 39	1335	1334	120	20.1	Header	750	1670	Header	793	188.3	214	196	136	BEOW
Hoor 35		1255	121	25.1	Header	20.7	144.8	Header	29/3	189.3	210	178	174	694%
Hoar 37	1603	1,3188	133	27.8	Header	797	403.7	Header	293	189.3	269	433	2869	96366
Floor 36		1-03-0	106	588	Header	16.4	1225	Heades	283	1984	502	157	751	50776
Flaor 35	1877	1601	107	37.0	Header	30'0	6001	Header	283	1691	218	210	210	6578
Floor 34		1307	901	34.1	Header	16.4	1225.	Header	29.3	1893	306	152	152	44506
Floor 13	2219	1815	107	36.3	Header	30.0	5081	Header	79.1	1893	219	210	210	579%
Floor 32		1551	107	30.4	Header	16.4	122.5	Header	29.3	189.1	205	187	152	395%
Floor 31	7461	2031	110	40.6	Header	36.2	2003	Header	79.3	189.3	226	230	.526	92509
Floor 30		2141	110	42.8	Header	707	14430	Header	293	109.3	210	174	1/4	40798
Floor 29	2703	7249	109	45.0	Header	30.0	180.9	Header	293	189.3	219	210	210	467%
Floor 28		2358	108	47.2	Header	164	1225	Header	283	189.3	208	152	152	322%
Floor 27	2945	2467	109	49.3	Header	30.0	180.9	Header	293	1893	219	210	210	426%
Hoor 26		2577	110	515	Header	16.4	122.5	Header	29.3	1893	208	152	157	295%
Floor 25	3187	2697	110	-53.7	Header	390	180.9	Header	283	189.1	219	210	210	351%
Floor 24	100	.2795	108	653	Header	16.4	1225	Header	. 25.0	167.0	183	1431	148	264m.
Flass 23	3429	7941	145	SAR	Header	36.2	2063	Header	25.0	167.0	202	234	203	346%
Floor 77		20165	144	81.7	Header	20.7	1443	Header	20.7	144.8	591	165	165	2639%
Floor 21	3704	3230	145	64.6	Header	36.2	2000	Header	707	144.0	101	230	181	2,009%
Flast 20		3344	114	629	Header	16.4	122.5	Header	20.7	144.0	191	143	143	21406
Hoor 19	3980	3458	115	69.7	Header	0'06	180.9	Headyr	20.7	1443	175	202	175	253%
Floor 18		3573	3115	71.5	Header	16.4	172.5	Header	20%	1448	161	143	343	200%
Floor 17	4168	3688	115	73.8	Header	30.0	1809	Header	20.7	144.8	175	202	175	237%
Fluor 16		3809	121	762	Header	16.4	122.5	Header	25.0	167.0	133	148	148	154%
floor 15	4505	3939	130	78.8	Header	30.0	6184	Seated	42	42.	72	223	72	956
Floor 14		4040	102	80.8	Header	16.4	1325.	Header	16.4	122.5	139	139	139	172%
Floor 13	4697	4143	103	87.8	Header	30.0	180.9	Header	16.4	122.5	153	197	.153.	104%
Hwy 12		4245	102	84.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	164%
Floor 11	4859	4349	103	87.0	Header	0.00	1809	Header	164	122.5	151	161	153	1 74/6
Floor 10		4452	104	0.68	Header	16.4	122.5	Header	16.4	172.5	139	1389	139	156%
Floor 9	5081	4556	104	91.1	Header	30.0	190.9	Header	16.4	122.5	131	161	153	167%
Root 8		4659	303	93.2	Header	164	1225	Headen	16.4	122.5	139	139	139	14996
Hoor 7	4896	4270	-380	85.4	Double Fire	519.0	519,0	Double Fin	276.0	2768	967	7967	796	93206
Hoore		4214	-55	84.3	Truss Chx	NC.	MC	Truss Chx	MC	MC	MC	NC	NC	>100
Hoor 5	5243	4104	.110	87.1	Truss Clur	NC	MC	Truss Clux	MC	MC	NC	WC.	NC	>100
Roce 4		4318	214	88.4	Juns Cax	274	NC.	huss Cox	No.	NC	NC.	WC.	MC.	> 100
Floor 3	5693		1)				+		, and	10	4	2	÷	
Floor 2		1	Y			X	X			ĭ	North Control of the	-		4

Hoor	As count Awaii	Orthodox Cont.		Property Present		The last of the last	Trained concerns to the contract of the contra	DESCRIPTION OF THE PARTY OF	The Market of the Land of the			Constitution that the stand	Contraction Parket	
Inc	(Camulative)	(Cumulative)	(Intrements)	Capabry (7%)		Morth Connection			South Connection		North in Tension +	South in Tension +	I'mwideli by the connections unity	Percentage of Reg.
	Repl	(Kipl	[Kip]	(Kip)	Conn Type	T (Nip)	C leip]	Conn Type	This	C(kip)	South in Comp. [ktp]	North in Comp. (kp)	Minimum (kip)	Bracing Capacity
Roof		105	105	2.1	Knife	60	630	Knife	80	63.0	69	689	69	330346
Flast 47	11	170	-99	3.4	Knife	10.3	105.0	Knife	8.1	840	34	113	3	2774%
Floor 46		254	.04	2.1	Knife	12.4	126.0	Kaife	8.1	84.0	98	104	98	185649
Hoor 45	390	334	00	6.7	Knife	12.4	126.0	Knite	6.0	63.0	75	132	75	1120%
Floor 44		396		8.6	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	947/46
Fluor 43	513	436	318	9.7	Kulfe	12.4	128.0	Kolfe	6.0	63.0	75	137	8	BSAN
Fluor 52		518	62	10.4	Knife	124	1260	Knife	.09	610	7.5	102	75	728%
Floor 41	529	255	38	11.1	Knife	12.4	1260	Knife	8.0	018	75	192	52	681%
Ekor 40		646	343	17.9	Kaile	124	1280	Knife	60	83.0	5/	132	32	583%
Elizor 39	737	.715.	69	14.3	Knife	13.1	126.0	Knife	0.9	63.0	76	132	32	5329A
Floor 38		703	89	157	Knite	13.1	126,0	Knife	6.0	63.0	1/6	132	9/	ASSW
Floor 37	848	8884	10	173	Knife	9.6	147.0	Kinte	8.0	83.0	2	153	73	421%
Floor 36		918	55	18.4	Knife	103	106.0	Knife	6.0	63.0	13	111	22	3185%
Fluor 35	988	976	57	19.5	Knife	10.3	0.501	Knife	6.0	63.0	5.3	111	73	379%
Floor 34		1002	95	20.6	Knife	10.3	0.501	Knife	0.0	63.0	7.0	111	73	35506
Floor 13	1030	1009	57	21.8	Kaife	10.1	1050	Knife	99	019	77	111	73	336%
Floor 32		1146	-95	32.9	Knife	103	105.0	Knife	0'9	63.0	73	1115	22	320%
Fluor 31	1174	1209	63	242	Knife	10,3	105.0	Knife	0'9	63.0	73	111	73	302%
Floor 50		12/6	19	355	Knife	10.1	105.0	Knife	140	2100	220	119	119	467%
Floor 29	1404	1,024	69	785	Knife	10.3	1050	Knife	8.0	83.6	7.3	-111	13	277%
Flast 28		(372	48	33.4	Kniře	10.3	105.0	Knife	0.9	63.6	22	-111	73	287%
Elect 27	1472	1421	49	20.4	Knife	103	105.0	Knife	6.0	63.0	73	111	73	258%
Hoor 26		1469	48	29.4	Knife	10.3	105.0	Knife	0.9	63.0	12	111	73	24996
Floor 25	1540	1518	69	30.4	Karte	10.3	105.0	Knife	6.0	93.0	73	111.	7.3	241%
Flace 24	1.00	1566	48	31.3	Knife	10.3	105.0	Knife	6.0	979	23	111	E.	234m.
Fluor 23	2545	1677	70	32.7	Knife	8.1	840	Knife	12.6	189.0	197	97	16	79Ms.
Floor 22	-	1690	53	318	Knife	8.0	63.0	Knife	8.0	63.0	639	699	69	204%
Floor 21	7603	1744	25	34.9	Kalfe	6.0	63.0	Knife	0.0	69.0	638	69	69	1966
Floor 20		1706	42	357.	Kulle	60	63.0	Knife	60	63.0	-65	69	69	19396
Hoor 19	2700	1001	44	300	Knite	6,0	63.0	Knife	6.0	000	69	69	60	130%
Floor 18		1874	43	375	Knife	6.0	63.0	Knife	0.9	63.0	69	69	69	184%
Flant 17.	2802	1919	45.	38.4	Knite	60	63.0	Knife	6.0	83.0	69	69	69	180%
Fluor 16		1988	49	39.4	Knife	6,0	63.0	Knife	6.0	63.0	619	69	699	17986
floor 15	3015	2026	59.	80.5	Seated	42	42	Knife	6.0	939	105	48	48	11690
Floor 14		2058	31	41.2	Knife	6.0	63.0	Knife	0'9	63.0	69	69	69	168%
Floor 13	3063	2009	31	41.0	Knife	6.0	63.0	Knife	6.0	63,0	69	69	69	165%
Histor 12	2000	2121	31	424	Knife	6,0	630	Knife	09	63.0	63	63	69	163%
Blear 11	3151	2154	33	43.1	Kulle	6.0	63.0	Knife	60	63.0	69	69	60	160%
Floor 10		2185	31	43.7	Knife	60	63.0	Knife	6.0	063	69	69	69	158%
Floor 9	3219	2218	33	44.4	Kaite	6.0	63.0	Knife	6.0	63.0	60	69	69	156%
Floor B		2250	32	45.0	Knife	6.0	63.0	Knife	6.0	0.00	69	69	69	153%
Hote 7	3337	2175	-74	43.5	Truss Clox	NC	NC	Trust Cho.	NC	NC	NC	NC	NC	>100
Hoore	-	2213	38	44.3	Hin	86.5	96.5	Fin	121.1	121.1	208	208	802	46946
Hoor 5	3497	1692	-532	33.6	Truss Cox	NC	NC.	Truss Cinc	MC	NC	NC.	WC.	MC	>100
Floor 4		1071	52	34.1	Į.	989	888	Ī	1211	121.1	208	2008	208	80908
Floor 3	3664	22.12	525	14.6	Truss Onc	NC	NC.	Truss Cnx	NC	NC	NC	MC	NC	>100 >100
Hoor 2		10000	9.90								0.00			

gn axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing f

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	Column brands	Column I nearly	Column Iread	Caracido Dithi		History and and	Transport of the last of the l	find an arrange of the state of	Endred and			(Provided by the C	Provided by the Connections Only	
9	Camulative	(Cumulative)	(Increments)	(Kind	Chain faor	West Connection	C leal	Coon No-	East Connection Third	C (biol	West in Tension + East in Como (No)	East in Tension a West	Minimum (Kip)	Percentage of Reg.
Roof	Habi	105	105	2.1	Hender	16.4	122.5	Header	16.4	172.5	139	139	138	SRASIN
Flast 47	11	170	-98	3.4	Header	2007	144.8	Header	338	211.6	232	178	178	8355E
Floor 46		254	.04	13	Header	25.0	0291	Header	202	144.8	170	160	170	334090
Hoor 45	390	334	00	6.7	Header	29.3	1863	Header	20.7	1440	174	210	174	2005/06
Floor 44		396	64	8.6	Header	16.4	122.5	Header	707	144.8	161	143	143	1799N
Eluor 43	513	436	38	9.7	Header	707	144.8:	Header	20.7	14431	185	165	165	1897%
Flior 42		518	62	10.4	Header	16.4	1225.	Header	207	144.6	191	143	143	13624
Floor 41	529	553	38	111	Header	207	144.8	Header	207	1443	165	165	165	149549
Floor 40		646	343	12.9	Header	16.4	1225	Header	207	144.8	161	143	143	1100990
Fluor 39	737	715	69	14.3	Header	20.7	144.5	Header	267	144.8	165	165	165	1157%
Floor 38		703	89	15.7	Header	16.4	122.5	Header	20.7	144/8	161	143	143	914%
Floor 37	848	884	61	17.3	Header	42.2	258.1	Header	20.7	1443	197	217	187	108280
Floor 36		818	55	184	Header	16.4	1225	Header	20.7	14421	161	143	143	77646
Flaor 35	986	976	25	19.5	Header	30.0	180.9	Header	20.7	144.8	175	202	175	MS6B
Floor 34		1002	56	20.6	Header	16.4	1225	Beader	20.7	1440	161	143	143	69469
Floor 13	1030	1,009	25	21.8	Header	30.0	5981	Header	707	144.0	175	202	175	80298
Floor 32		1146	.95	22.9	Header	16.4	122.5	Header	207	144.8	191	143	143	625%
Huor 31	1174	1209	-63	242	Header	293	189.3	Header	20.7	14433	174	210	174	720%
Floor 30		17/6	19	255	Header	41.7	758.1	Header	707	144.8	107	277	187	71598
Floor 29	1404	1324	69	785	Header	30.0	1809	Header	20.7	1448	175	202	175	660%
Flaor 28		(37)	48	33.4	Header	164	1225	Header	707	1448	191	143	143	5226
Eleot 27	1472	1421	49	26.4	Header	30.0	1809	Header	20.7	144.8	175	202	175	61508
Hoor 26		1469	48	29.4	Header	16.4	1225	Header	20.7	1448	161	143	143	4884
Floor 25	1540	1518	69	30.4	Header	390	189.9	Header	. 29.7	1448	175	202	175	578%
Flace 24	1000	1566	48	31.3	Header	16.4	1225.	Header	.207	14433	191	143	143.	4574
Flast 23	2545	1637	70	32.7	Header	30.0	1309.9	Header	42.7	256.1	206	223	223	682°ss.
Floor 22	-	1690	53	318	Header	16.4	1725	Header	20.7	144.8	191	DA3	143	424%
Floor 21	7603	1244	Ž,	34.9	Header	30.0	6001	Header	707	1440	175	202	175	507%
Floor 20		1706	42	357.	Header	16.4	122.5	. Header	164	122.5	139	109	(39	36096
Floor 19	2708	1001	44	34.6	Header	0'06,	1809	Headyr	16,4	1725	153	161	153	417%
Floor 18		1674	63.	37.5	Header	16.4	7225	Header	36.6	172.5	139	139	139	371%
Flant 17.	2802	1919	45.	38.4	Header.	30.0	189.9	Header	16.4	122.5	153	197	153	397%
Fluor 16		1968	49	39.4	Header	16.4	1225	Header	75.0	167.0	1303	148	148	375%
Hoor 15	3015.	2026	-88	80.5	Header	30'0	-6884	Header	33.6	2116	242	215	215	6299sh
Floor 14		2058	31	41.2	Header	12.2	1002	Header	16.4	122.5	135	117	117	28396
Floor 13	2062	2009	31	41.0	Header	30.0	188.9	Header	16.4	122.5	153	197	153	365%
Histor 12	-	2121	31	424	Header	16.4	122.5	Header	16.4	122.5	139	139	139	326%
Hear 11	3151	2154	13	43.1	Header	0/06	1809	Header	16.4	122.5	153	161	153	354%
Floor 10		2185	5	43.7	Header	16.4	122.5	Header	164	172.5	139	139	139	310%
Floor 9	3219	2218	33	44.4	Header	30.0	1993	Header	16.4	122.5	151	161	153	34496
Roor B		2250	32	420	Header	16.4	179.5	Header	16.4	1225	139	138	139	3090%
Floor 7	3337	2175	-74	43.5	Header	20.7	144.5	Header	16.4	122.5	143	191	143	329%
Hooris		2213	38	44.3	Header	621	129.9	Header	16.4	127.5	1940	146	140	3178
Hoor 5	3497	1687	-532	33.6	Header	42.2	256.1	Header	42.2	756.1	298	362	298	8874
Roce 4		1/0/1	7.8	34.1	Header	4.8	17.8	Header	33.6	2116	218	112	112	32.7%
Boor 3	3664	22.12	525	44.6	Header	33.6	211.6	Header	13.6	211.0	245	245	245	54996
Hoor 2		2 2202 3:0 45.2 Weder 25.0 (67:0 Weder (6.4 (22.5 ) 48 (93.0	30	45.2	Header	25.0	167.0	Header	4.91	122.5	148	183	148	32698

Floor	Column heads	Column I cash	Column Ireal	Canarily Diffet		AS-INSIGE	AS-Designed Axial Capacity of Bracing Connections [Adv]	A DIALING SAMESAN	forbyl sau			(Provided by the 2	Provided by the Connections Only	
	(Camulative)	(Cumulative) (Kpl	(Increments)	Kipl	Conn. Type	North Connection T [kip]	C Ripl	Conn. Type	South Connection 7 [kip]	Clapl	North in Tension + South in Comp. [ktp]	South in Tension + North in Comp. (kgp)	Minimum (Rip)	Percentage of Req. Bracing Capacity
Rost		339	39	0.8	Knife	8.0	63.0		0	0	æ	63	16	74596
Floor 47	87.	582	46.	-12:	Knife	8.1	840	)	0	0	30	84	60	47.786
Floor 46		170	84	334	Kaife	1%	84.0	ı	0	0	0	84	8	240%
Hoor 45	485	253	84	5:1	Knife	60	63.0		0	0	9	63	9	11904
Floor 44		331	78	88	Knife	6.0	63.0	X	0	ō	9	63	16	9016
Floor 43	674	361	30	7.7	Knife	6.0	63.0	-	0	0	. 9	63	.9	B3Wk
Floor 52		454	93	9.1	Knife	6.0	63.0		.0	0	9	63	9	9000
Floor 41	298	493	36	66.	Knife	8.0	63.0		0	0	9	19	(s)	619
Floor 40		265	104	11.9	Knife	6.0	63.0	,	0	0	15	63	œ	NOS
Floor 39	1050	929	79	13.5	Knife	0.9	63.0		.0	0	.9	63	ú	4408
Hoor 38		756	90	15.1	Knife	6.0	63.0		.0	0	10	63	iò	4004
Hoor 37	1238	836	-08	18.7	Knife	8.0	63.0		0	0	9	63	8	THEM
Floor 35		916	300	18.3	Knife	60	63.0		0	.0	4	63	- 6	13%
Fluor 35	1426	987	181	19.8	Knife	6.0	63.0	À	0	0	9	63	19	300
Floor 34		1078	100	21.6	Knife	6.0	63.0	3	.0	0	. 9	63	.9.	NAZZ.
Floor 33	1614	1159	RI	23.2	Knife	99	63.0		0	0	9	63	9	2006
Floor 32		1241	11.2	24.8	Knife	6.0	63.0	2	0	0	19	63	9	240
Floor 31	1802	1327	.90	26.5	Kaife	6,0	63.0		0	0	9	63	9	27%
Floor 30		1415	69	2863	Knife	140	210.0		0	0	14	210	14	Strills
Floor 29	2069	1489	73	29.8	Knife	6.0	83.0	V	. 0	0	Œ	63	28	2004
Floor 28		1562	74	312	Kniře	6.0	63.0	,	0	0	9	63	ig.	1846
Floor 27	2239	1636	73	32.7	Knife	6.0	63.0	£	0	0	10	63	io	18%
Hoor 26		1709	74.	34,2	Knife	6,0	63.0	<	0	0	9	63	9	18nl.
Floor 25	2409	1783	74	35.7	Knife	6.0	63.0	1	.0	0	9	63	9	17%
Floor 24		1854	7.1	37.1	Xnife	6.0	63.0	x	.0	0	19	63	6	164W
Flisse 23	2579	1946	93.	30.9	Knife	11.2	1650		.0.	0	11	168	11	29%
Floor 22		2026	. 6/	40.5	Knife	6,0	63.0	V	0	0	9	63	ian .	15%
Floor 21	2835	2105	. 79	42.1	Knife	60	63.0	X	0	0	9	63	9	144
Floor 20		2169	49	424	Knife	60	63.0		0	0	40	63	9	1406
Hoor 19	2909	2233	64	44.7	Knite	6,0	63.0		0	0	. 9	63	19	1,july
Floor 18		7227	69	659	Knife	6.0	63.0		0	0	9	63	9	130
Floor 17	3127	2361	64	47.2	Knife	6.0	63.0	,	0	0	9	63	9	13%
Fluor 16		2425	64	48.5	Knife	6.0	63.0	j.	0	0	ug.	63	œ	1246
Floor 15	3265	2489	64	49.8	Kaife	0'9	63.0		0	0	9	63	.0	120
Floor 14		2554	59	51.1	Knife	6.0	63.0	X	0	0	- 53	63	9	龙
Floor 13	3403	2619	99	52.4	Knife	6.0	63.0	,	.0	ō	9	63	9	124
Hoyr 12		2683	65	53.7	Knite	0'9	630	X	0	0	æ	63	6	1198
Floor 11	3541	2748	- 59	055	Knife	60	63.0		0	0	ie.	63	ò	11%
Floor 10		2814	99	583	Knife	6.0	630		0	0		63	ie:	11990
Floor 9	3679	2879	99	87.8	Knife	6.0	63.0	,	0	0	CE .	63	ú	100
Floor 8		2944	.99	6119	Knife	6.0	0.03		. 0	0	9	63	iá	1048
Hoor 7	3857	2740	-204	54.8	Truss Onx	NC	NC	Knite	6.0	63.0	NC	NC	NC	>100
Hoor 6		2856	116	57.1	Hn	121.1	121.1	×	0	0	121	121	121	212%
Floor 5	4176		.550	46.1	Truss Clur.	NC	MC		0	0	NC	MC	NC.	>100
Floor 4		2156	150	43.1	E.	121.1	1711	X	0	0	121	121	121	281%
Hoor 3	4460	2855	689	57.1	Truss Onx	NC	NC.		0	0	MC	NC	NC	>100
Floor 2		0000	235	718	Fier							100		

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INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 72 (EAST - WEST) C6.30

Roor	As built Axial Column trads.	Column topils	Column Ired	Capacity (2%)		AS-Design	As Designed Axial Capacity of Bracing Connections (Kips)	A PACING LOGHECTES	ins [Augus]			(Provided by the Connections Only)	unections Only)	
	(Camulative) (Kpl	(Cumulative) (Kipl	(Increments)	(Kip)	Onn. Inc	West Connection 1 Dopl	C (bp)	Conn Pype	East Connection 7 (kip)	C [kip]	West in Tension + East in Comp. [htp]	East in Tension a West in Comp. [kp]	Minimum (Rip)	Percentage of Req. Bracing Capacity
Roof		120	39	80	Hender	16.4	1725	Header	16.4	172.5	139	133	138	17687%
Flast 47	87	885	- 99	1.7	Header	7987	144.8	Header	33.6	2118	232	328	178	10459%
Floor 46		170	.04	3.4	Header	37.3	213.4	Header	61.0	2007	338	275	275	8100%
Hoor 45	485	253	- 94	5.1	Header	20.7	144.8	Header	13,6	2116	232	170	1/8	3510%
Floor 44		331	78	99	Header	164	122.5	Header	33.6	211.8	728	188	156	2357%
Eluor 43	674	361	30	7.2	Header	No Info	No Info	Header	No tafo	Mohitio	No info	Moinfo	No info	
Fluor 42		454	93	9.1	Header	No info	No Info	Header	33.6	2116	No info	tho info	Mo info	,
Floor 41	1962	493	5E	6.5	Header	No lafe	No Info	Header	Notafo	No fofo.	Noinfo	Noinfo	Noinfo	
Floor 40		265	104	11.9	Header	No Info	No info	Header	13.6	211.6	No info	No info	No info	
Elmor 39	1050	94.9	79	13.5	Header	16.4	1225.	Header	336	2116	328	156	95)	1155%
Hoor 38		756	80	15.1	Header	16.4	1225	Header	33.6	211.5	228	3%	156	103346
Floor 37	1238	836	- 00	187	Header	18.4	122.5	Header	33.6	2118	228	158	156	934%
Floor 36		916	260	18.3	Header	164	172.5	Header	33.6	2116	228	156	156	40,746
Flaor 35	1426	987	180	19.9	Header	16.4	1225.	Header	33.6	2116	228	.951	.951	783%
Floor 34		1078	10	21.6	Header	16.4	1225	Header	33,6	2116	228	.951	951	724%
Floor 13	1614	1159	RT	23.2	Header	16.4	1225	Header	33.6	2116	228	35	156	9339
Floor 32		1241	10.2	24.8	Header	16.4	122.5	Header	13.5	211.6	228	158	156	529W
Floor 31	2081	1327	96	26.5	Header	16.4	1725	Header	33,6	2116.	223	196	.951	580m
Floor 30		1415	69	28.3	Header	13.6	2116	Header	13.6	2116	216	245	245	(BSS)
Floor 29	2069	1489	13.	29.8	Header	164	172.5	Header	31.6	2118	228	.158	156	524m
Fluor 28		1562	74	31.2	Header	16.4	122.5	Header	33.6	211.8	228	(50)	951	500%
Eleor 22	2239	1636	73	32.7	Header	16.4	122.5.	Header	33.6	2116	228	156	156	477%
Hoor 26		1709	74.	34,2	Header	16.4	122.5	Header	33,6	2116	228	156	156	457%
Floor 25	2409	1783	. 74	35.7	Header	16.4	122.5	Header	33.6	2116	228	156	156	438%
Fluor 24		1854	33	37.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	379%
Fluor 23	2579	1946	93.	38.9	Header	16.4	122.5	Header	33.6	2116	228	351	.126	4010s.
Floor 22		2026	79	40.5	Header	164	172.5	Header	16.4	1775	139	87	139	343%
Floor 21	2035	2105	79	42.1	Header	16.4	122.5-	Header	16.4	122.5	138	(38)	139	330%
Floor 20		2109	55	43.4	Header	16.4	122.5	Header	16.4	122.5	135	139	130	320%
Floor 19	2909	2733	64	44.7	Header	16.4	172.5	Header	16,4	172.5	139	139	139	331%
Floor 18		2207	19	45.9	Header	16.6	172.5	Header	16.6	772.5	139	139	139	302%
Floor 17	3127	2361	64	47.2	Header.	164	172.5	Header	16.4	1225	139	139	139	294%
Fluor 16		2475	64	48.5	Header	16.4	1225	Header	16.4	5221	139	139	139	288W
floor 15	3265	2489	64	868	Header	16.4	1225	Header	16.4	1225	139	138	139	27906
Floor 14		2554	529	51.1	Header	12.2	1002	Header	16.4	122.5	135	117	117	228%
Floor 13	3403	2619	65	52.4	Header	16.4	122.5	Header	16.4	122.5	109	139	139	265%
Hear 12		2603	69	53.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	259%
Bear 11	3541	2748	65	980	Header	16.4	122.5.	Header	16.4	122.5	139	139	139	753%
Floor 10		2814	919	593	Header	164	122.5	Header	16.4	172.5	119	139	139	247%
Floor 9	3679	2879	985	87.6	Header	16.4	1225	Header	16.4	1225	138	139	139	24196
Floor B		.2944	99	603	Header	164	1225.	Header	164	122.5	139	138	139	236W
Floor 7	3857	2740	-204	54.8	Header	72.1	194.9	Header	72.1	1943	267	797	292	4876
Floor 6		2856	116.	57.1	Truss Chic	0	0	Truss Chx	.0	0	MC	NC	NC	>100
Hoor 5	4178	2306	- 550	46.1	Truss Clux	0	0	Truss Clox	0	0	NC	WC	MC	>100
Roce 4		2156	180	43.1	huss Chix	0	o	Juns One	0	ó	W.	W.	NC	3.100
Floor 3	4460	2855	580	57.1	Truss Chx	0	0	Truss Chix	0	0	NC.	MC	NC	>100
Hoor 2		3550	235	2.4.0	Technical Contra									

e North-Sout innections On	Minimum	18	10	12.	10	10		10	Í	10	10	10	10	10	10	10	10	10	10
fotal Bracing Capacity in the Provided by the Con	South in Tension + North in Comp. [kip]	182	10	12.	10	10	Y	10		10	10	10	10	10	10	10	10	10	10
	South in Comp. [ktp]	7.5	105	126	105	105	7	105		105	105	105	105	105	105	105	105	105	106
	Chipl	63.0	105.0	126.0	105.0	105.0	0	105.0	0	105.0	105.0	105.0	105.0	105.0	1050	105.0	105.0	105.0	105.0
[Kips]	outh Connection 7 [kip]	8.0	10.3	12.4	10.3	10.3	0	16.3	0	103	10.3	10,3	16.3	103	10.3	10.3	10.1	10.3	101
As-Designed Axial Capacity of Bracing Connections [Kips]	Conn. Type	Knite	Knife	Knife	Knife	Knife		Knife		Kniře	Knife	Knife	Kinfe	Knife	Knife	Knife	Knife	Knife	Koile
Axial Capacity of	C Ripl	128.0	0	0	0	0	0	.0.	0	0	0	0	.0	0	.0	.0	0	0	
As-Designed	North Connection T Kipl	12.4	0	0	0	0	0	. 0	0	-0	0	0	- 0	0	.0	0	. 0	.0	.0.
		Knife	,	ı	,	X	,			,			,		7			5	
Reg Conn Axial Capacity (296)	Kipl	2.0	2.8	6.5	8.0	1171		13.6		18.3	18.7	21.0	23.4	25.7	28.1	30.5	32.9	35.3	224
Design Axial Column Iread	(Increments)	101	.46:	178	911	113		127		135	117	111	118	118	118	120	120	120	130
Design Axtal Column Loarls	(Cumulative) (Kipl	101	147	325.	441	554	- 1920	681	100	818	933	1090	1168	1287	1408	1526	1646	1766	Sept.
As built Axial Column brads.	(Camulative)		125		517		12211		11.00		1448		1753		2068		2378		THESE.
Roor		Read	Flast 47	Floor 46	Hoor 45	Floor 44	Thurst 3	Floor 42	Floor 41	Floor 40	Fluor 39	Hoor 38	Floor 37	Floor 36	Flaor 35	Floor 34	Floor 33	Floor 32	Burn 21

The design axial capacity of each connection was determined based on the geometry and detailing provided in the latest corresponding Frankel Steel Limited steel shop drawing for that connection

Knite

8411

WTC7 Global Collapse Analysis – Appendix C Interior Column Stability Analysis Report 12 February 2010

C6.31

3928 4276 4592 5291

100	
O D	
Design Axial	
Design Axial	
As Failt Axial	THE PARK TION
Floor	CAROLL VI LOW

Floor	As built Axial	Design Axial Column Locals	Column Irea	Canarite Diffe		As-Design	As Designed Axial Capacity of Bracing Connections (Rips)	A PACING CORRECTION	is [Mps]			Postaded by the Connections Dated	Appendicus Caled	
	(Cumulative)	(Cumulative)	(Increments)	Kipl	Conn Type	West Connection T Kipl	C Riol	Conn. Pape	East Connection 7 (kip)	C (kip)	West in Tension + East in Comp. (Mp)	East in Tension a West in Comp (kp)	Minimum (kip)	Percentage of Req. Bracing Capacity
Read		101	101	2.0	Header	20.7	144.8:	Header	20.7	1448	1665	165	185	8184%
Flax 47	125	147	46.	2.8	Header	707	144.8	Header	207	144.8	165	165	165	- 58339W
Floor 46		325.	178	.5'9	Header	61.0	2007	Header	111.9	395.3	457	413	413	634696
Hoor 45	517	441	116	8.0	Header	293	1893	Header	25.0	0791	196	214	196	2220%
Floor 44		654	113	1171	Header	29.3	(89.3	Header	25.0	167.0	198	214	196	1773%
Thurst 3	13211	588				0	0		0	0	×.			
Floor 42		681	127	13.6	Header	293	1993	Header	25.0	167.0	361	214	.196	144196
Also 41	1510	E893				0	0		0	0				
Floor 40		816	135	18.3	Header	29.3	1883	Header	25.0	167.0	198	214	196	1203%
Fluor 39	1448	933	117	18.7	Header	283	1893	Herader	25.0	167.0	361	214	1967	105208
Floor 38		1050	117	21.0	Header	29.3	189.1	Header	25.0	167.0	196	214	196	935/6
Floor 37	1753	1168	118	23.4	Header	293	189.3	Header	25.0	187.0	198	214	196	840%
Floor 36		1287	119	787	Header	283	189.3	Header	250	167.0	180	214	.981	26296
Flaor 35	2068	1406	119	28.1	Header	29.3	1893	Header	25.0	0791	961	214	.961	69896
Floor 34		1526	120	30.5	Header	293	1893	Header	25.0	167.0	961	214	.961	64396
Floor 33	2378	1646	120	32.9	Header	79.1	1893	Header	25.0	167.0	361	214	196	98265
Floor 32		1766	120	35.3	Header	293	189.3	Header	25.0	167.0	196	214	196	\$55W
Floor 31	2660	1600	120	37.7	Header	293	1893	Header	25.0	167.0	196	214	.1961	521%
Floor 30		2002	121	40.1	Header	162	109.3	Header	25.0	167.0	196	214	196	4600%
Floor 29	2998	2128	121	428	Header	793	1893	Header	250	167.0	生	214	1961	A61%
Floor 28		2248	121	45.0	Header	29.3	189.3	Header	25.0	167.0	196	214	961	43.7%
Floor 27	3300	2370	121	47.4	Header	293	1693	Header	25.0	167.0	196	214	.961	414%
Hoor 26		2462	122	49.8	Header	203	1893	Header	25.0	167.0	196	214	196	394%
Floor 25	3618	2614	122	.523	Header	29.3	1839.3	Header	25.0	167.0	198	-214	196	376%
Floor 24	1000	.2740	126	54.8	Header	052	167.0	Header	336	2115	237	301	701	366%
Fluor 23	3928	2687	147.	57.7	Header	25.0	167.0	Heater	33.6	2116	333	201	201	348%
Floor 77		1057	141	808	Header	20.7	144.8	Header	13.6	211.6	237	1/18	178	295%
Floor 21	4276	3168	141	63.4	Knife	10.3	1050	Seared	42	43	8	147	55	10.70
Floor 20		3264	116	65.7	Knife	103	105.0	Sented	42	45	25	147	- 25	800
Floor 19	4592	3400	116	600.0	Knite	10.3	1050	Scaled	42	47	25	147	25	442
Floor 18		3816	3116	70.3	Knife	10.3	105.0	Seated	45	45	25	147	2/5	nut 6
Flore 17	4882	3633	117.	72.7	Knife	10.3	105.0	Seated	-45	42	25	147	52	1997
Fluor 16		3756	124	75.1	Kaife	124	126.0	Seated	42	42	64	1638	84	17/04
Floor 15	5291	3886	130	111	Seated	42	42	Seated	42	42.	84	84	8.4	10698
Floor 14		3861	105	79.8	Knife	10.3	105.0	Seated	42	43	55	147	65	10509
Floor 13	5553	4096	305	613	Knife	10.3	105.0	Seated	43	43	. 52	147	28	1642
Heyr 12		4501	106	84.0	Knite	10.3	105.0	Scaled	42	2.6	25	147	25	10,50
Floor 11	5815	4307	106	86.1	Kuife	103	1050	Sealed	42	42	23	147	25	910
Floor 10		4415	108	8.03	Knife	103	1050	Seated	42	45	25	147	25	1697
Floor 9	6077	4521	107	90.4	Kaife	103	1050	Sexted	42	42	25	147	25	2005
Floor 8		4628	107	92.6	Kaife	10.3	105.0	Sented:	42	42	25	147	25	9000
Hoor 7	8411	4340	788	868	Double Fire	276.8	2768	Knife	40'8	488.4	761	310	318	366%
Hoor 6		4352	12	87.0	Truss Onc	NC.	MC	Nothfo	No Info	No Info	No info	No liifo.	No info	×
Floor 5	8206	6615	2263	132.3	Truss Clor.	NC	MC	Truss Clux	NC	MC	WC.	WC.	MC	>100
Floor 6		6786	180	135.9	Jruss Clux	MC	NC	Knife	20.9	2100	MC	W.	NC.	>100
Hoor 3	9710								í		4			
Floor 2			)											

**Guy Nordenson and Associates** 

INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 74 (NORTH - SOUTH)

livel	Column Trads	Column Insuls	Column I carl	Caracity Diski		As-Design	ed Axial Capacity c	As Designed Axial Capacity of Bracing Connections (Kips)	ns [kips]			Possided by the Connections Only	(Powded by the Connections Only)	
	(Camulative)	(Comulative)	(Increments)	(Kns)	Conn fyre	Morth Connection 7 [Ki6]	C Biol	Conn. Pype	South Connection 7 (kin)	C (kiel	North in Tension + South in Comp. (htp.)	South in tension +	Minimum (Rip)	Percentage of Reg. Bracing Capacity
Road		119	119	7.4	Knife	81	88.0	Knife	80	830	7.1	98	11	SHARE.
Flast 47	.76	180	27	3.8	Kaife	10.3	105.0	Knife	0.0	83.0	123	311	73	1928%
Floor 46		259	-69	5.2	Knife	12.4	126.0	Kaife	0'9	63.0	.35	132	75	145400
Hoor 45	330	329	70	99	Knife	10.3	1050	Knite	6.0	019	77	111	73	1112%
Floor 44		395	.88	1.9	Knife	10.3	0.201	Knife	6.0	63.0	7.3	111	- 13	52 PW
Thursd:	195.	200		,		.0.	0		0	0		1		
Bisor 42		475	99	9.8	Knife	10.3	1050	Knife	6.0	63.0	73	111	73	77196
3 lice 41	164	(1)				0	0		D	0				
Floor 40		547	87	112	Kalfe	10.3	105.0	Knife	0.9	0.08	12	111	10	655%
Fluor 39	1007	634	72	127	Knife	10.3	105.0	Knife	0.9	963	.73	111	73	5789k
Hoor 38		.90/	72	14.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	77	519%
Floor 37	1259	612	73	15.8	Knife	10.3	105.0	Kinfe	8.0	83.0	73	111	13	450%
Floor 36		2883	74	171	Knife	103	105.0	Knife	6.0	63.0	123	111	- 78	430%
Flaor 35	1491	927	74	18.5	Knife	10.3	0501	Knife	0.9	83.0	2.3	111	73	395/8
Floor 34		1001	.75	200	Knife	10.3	105.0	Knife	0.0	63.0	2.0	111	73	36696
Floor 33	1723	9/01	75	215	Kaife	10.1	1050	Knife	9.0	610	7.7	III	73	340%
Floor 32		1111	.75	22.0	Knife	10.3	0.201	Knife	6.0	63.0	73	111	73	316%
Hoor 31	1955	1227	75.	24.5	Kaife	10,3	105.0	Knife	6.0	63.0	7.3	111	73	290%
Floor 30		1303	16	26.1	Knife	101	105.0	Kriffe	0.9	630	E.	111	73	2819h
Floor 29	2187	1379	76	278	Knife	10.3	105.0	Knife	8.0	83.0	73	-111	13	266%
Floor 28		1456	11	28.1	Kniře	10.3	105.0	Knife	6.0	63.6	73	-111	73	252%
Eleor 27	2419	1835	80	30.7	Knife	10.3	105.0	Knife	6.0	63.0	73	111	13	23996
Hoor 26		1615	90	32.3	Knife	10.3	105.0	Knife	6,0	63.0	12	111	73	227%
floor 25	2651	1695	80	33.9	Knite	10.3	105.0	Knife	6.0	0.08	73	111	7.3	216%
Flace 24		1768	72	354	Knife	124	126.0	Knife	6.0	63.0	22	132	75	213%
Hay 23	2973	1874	106	37.5	Knife	9.8	147.0	Knife	14.0	210.0	350	191	191	4309s.
Floor 22		1608	7.9	32.)	Knife	10.3	1050	Knife	8.0	630	n	111	13	189%
Floor 21	3345	1998	62	40.0	Kalfe	10.3	050	Knife	09	69,0	7.3	111	7.1	103%
Floor 20		2021	23	410	Knife	103	050	Knife	0.0	63.0	7.3	III	72	1700%
Hoor 19	3535	2103	53	42.1	Knite	10.3	105.0	Knife	0.0	63.0	13	111	E	174%
Hoor 18		2153	459	43.1	Knife	10.3	105.0	Knife	0.9	63.0	ZZ.	111	83	170%
Floor 17	3715	3503	-69	440	Kede	10.3	105.0	Knife	6.0	63.0	73	111	73	166%
Fluor 16		3228	54	45.1	Knde	12.0	126,0	Knife	6.0	63.0	75	137	76.	166W.
Hoof 15	3935	2317	61	863	Knife	12.6	0,634	Knife	0.9	63.0	3/6	195	22	16.70n
Floor 14	3	2359	45	47.2	Knife	0.0	62.0	Knife	0'9	63.0	69	69	69	146%
Floor 13	4105	2401	42.	40.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	344%
1 (epr 12		2443	42	40.9	Knife	6.0	63.0	Knife	09	63.0	63	69	69	1419%
Reor 11	4275	2485	42	49.7	Kuife	6,0	63.0	Knife	6.0	63.0	69	69	609	139%
Floor 10		252	47	808	Knife	0.9	63.0	Knife	6.0	63.0	69	69	69	136%
Floor 9	4445	2579	47.	51.5	Kalle	6.0	63.0	Knife	6.0	63.0	60	69	69	134%
Root 8	200	2626	-47.	52.5	Knife	0.0	0.09	Knife	0.9	0.00	69	69	69	13106
Hoor 7	8351	2605	-21	52.1	Truss Clox	NC	NC	Trust Crox	NC	NC	MC	NC	NC	>100
Floor 6		2689	- 44	52.0	Double Fift	103.8	103.8	Hin	121.1	121.1	225	775	332	42599
Hoor 5	5827	4612	1963	92.2	Jruss Cur.	MC	MC	Truss Clus	MC	MC	NC.	WC	NC.	>100
Roce 4		4844	32	92.9	Fetr (s.Augyle)	98.9	588	Fin (+Augle)	121.1	1211	208	208	208	224%
Boor 3	9226	5171	125	102.4	Truss Onc	NC	NC	Truss Cnx	NC	NC	NC	WC	NC	> 100
Hoor 2		5211	40	1042	Fin (+Angle)	101	1311	The Carbonian	-	-				

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INTERIOR COLUMN BRACING CAPACITY CATALOGUE - COLUMN 74 (EAST - WEST)

Roor	As built Axial Column buards	Design Axial Column Loads	Column Iran	Capacity (296)		Disables of	AS-Designed Axial capacity of Bracing Connections (April	a nacing connected	in Individual			(Provided by the Connections Univ.)	nnections Only)	
	(Camulative) (Kpl	(Cumulative) (Kpf	(Increments) (Kip)	Kipl	Conn. Type	West Connection T [kip]	C listel	Conn. Pype	East Connection 7 [kip]	C (kip)	West in Tension + East in Comp. [Mp]	East in Tension a West in Comp. (kp)	Minimum (Rip.)	Percentage of Reg. Bracing Capacity
Read		119	118	2.4	Header	16.4	1725	Heater	207	1448	181	143	143	6075/16
Floor 47	78.	180	311	3.8	Header	33.6	211.6	Header	707	144.8	178	737	-0.78	4694W
Floor 46		259	69	5.2	Header	20.7	144,8	Header	203	144,8	165	165	165	319266
Floor 45	330	329	70	99	Header	20.7	144.8	Header	20.7	144.8	165	165	165	253296
Floor 44		395	- 99	1.9	Header	707	144.8	Header	20.7	144.8	165	169	165	2095/46
Thurst.	195.	897		0		0	-0		0	0		-		
Floor 52		475.	99	98	Header	.207	144.8	Header	20.7	1443	166	165	165	17419h
Floor 41	1111	477		4		0	0		0	0				
Floor 40		5672	87	112	Header	70.7	144,8	Header	202	144.8	165	165	165	1473780
Fluor 39	1027	634	72	12.7	Header	207	144.5	Header	767	144.8	165	165	165	13050h
Hoor 38		106	72	14.1	Header	20,7	144,8	Header	20.7	144,8	165	165	165	1172%
Floor 37	1259	779	13	15.8	Header	20.7	144.8	Header	20.7	1448	165	165	185	1062740
Floor 36		883	7.4	171	Header	707	144.8:	Header	20.7	1442	165	165	.165	970%
Flaor 35	1491	977	74	18.5	Header	20.7	144,8	Header	20.7	144.8	165	165	165	MS SHR
Floor 34		1001	.75	200	Header	20.7	144.0	. Header	20.7	1448	165	591	165	826%
Floor 13	1723	1076	75	215	Header	70.7	144.8	Header	7.02	144.8	165	165	165	769%
Floor 32		1151	.75	23.0	Header	707	144.8	Header	20.7	144.8	165	591	391	719%
Floor 31	1985	1227	75.	24.5	Header	707	144.8	Header	20.7	14438	165	165	.165.	675%
Floor 30		1303	31	26.1	Header	707	144.8	Header	70.7	1448	165	165	165	6150
Floor 29	2187	1379	76	27.6	Header	70.7	144.8	Header	20.7	1448	165	166	165	800%
Floor 28		1456	11	28.1	Header	20.7	144.8	Header	20.7	1448	165	165	165	569%
Floor 27	2419	1535	90	30.7	Header	707	144.8	Header	20.7	144.8	165	165	165	53906
Hoor 26		1615	80	32.3	Header	20.7	144.8	Header	20.7	1448	165	165	165	51296
Floor 25	2651	1695	80	33.9	Header	7.02	144.8	Header	7.92	144.8	165	165	165	485%
Floor 24		1763	72	35.4	Header	7.07	144.8	Header	33,6	2115	232	178	178.	504%
Fluor 23	2973	1874	106	37.5	Header	61.1	320,1	Header	33.6	2116	273	384	273	727%
Floor 22		1908	139	30.7	Header	20.7	144.8	Header	20.7	1448	165	165	165	427%
Floor 21	0345	1998	62	40.0	Header	20.7	144.8	Header	20.7	1440	165	591	165	414%
Flast 20		1021	53	41.0	Header	16.4	122.5	Header .	20.7	144.8	161	143	143	34996
Floor 19	3535	2103	53	42.1	Header	16.4	177.5	Header	7.07	1443	191	143	143	3.40%
Floor 18		2153	-63	43.1	Header	16.6	172.5	Header	7.02	1448	191	143	343	3339
Floor 17.	3715	2202	-69	.44.0	Header	16.4	1225	Header	20.7	1448	161	143	143	325%
Fluor 16		3256	54	45.1	Header	75.0	167.0	Header	7.07	144.8	170	1881	170	376%
floor 15	3935	2317	69	863	Header	33.6	211,6	Header	20.7	1443	178	232	178	38SW.
Floor 14		2359	42	47.2	Header	16.4	122.5-	Header	20.7	1448	191	140	143	304%
Floor 13:	4105	2401	42.	46.0	Header	16.4	122.5	Header	20.7	144.8	191	143	.143	290%
Heyr 12		2443	42	46.9	Header	16.4	122.5	Header	70.7	144.8	161	143	143	293%
Floor 11	4275	2485	42	49.7	Header	16.4	122.5.	Header	20.7	1443	161	143	143	789%
Floor 10		25.02	47	808	Header	16.4	122.5	Header	20.7	1448	181	143	143	283%
Floor 9	4445	3579	47.	51.6	Header	16.4	1225.	Header	26.7	144.0	16.0	143	143	276%
Floor 8		2626	47.	62.6	Header	164	1225.	Header	707	14438	191	143	143.	2739h
Hote 7	8351	2605	-21	52.1	Knite	8.1	84.0	Knife	14.5	147.0	155	36	36	185%
Hoor 6		2689	44	53.0	Knife	8.1.	84.6	,	0:0	0.0	ll ll	84	69	1500
Hoor 5	1288	4612	1963	57.7	Knife	14.0	210.0	Truss Clux	JN.	MC	NC	WC.	MC	>100
Floor 4		4844	32	878	Knile	11.2	1680	×	0.0	0.0	11	1635	11	1768
Floor 3	9728	1215	125	103.A	Knife	412	1680		0'0	0.0	11	169	11	1199
Floor 2		5211	40	1040	Kolle		0.4.0							

(Camulative) (Rod Roof Rod Roor 47 85	The state of the s	Continue Liver	THE PROPERTY OF THE PARTY OF TH								the country of the country of the country of	STILL CHARGO STILLY	
-	_	(Increments)			Morth Connection			South Connection		North in Tension +	South in Tension +	The state of	Percentage of Reg.
		[Kip]	IKipI	Conn. Type	T Bipl	C lispl	Conn. Type	T Dkip]	Clkipl	South in Comp. [ktp]	North in Comp. [kip]	Minimum [kip]	Bracing Capacity
		85	12	Knife	8.0	63.0	1	0	0	rate and the same	63	*	822%
	105	41	.2.1	Kaife	6.0	63.0	,	- 6	.0	9	63	9	286%
Floor 46	207	102	90	Kaife	6,0	63.0		. 0	0	9	63	.9	14500
Hoor 45 39g	279	72	979	Knife	6,0	63.0		.0	0	9	63	9	106%
	350	7.1	7.0	Knife	6.0	63.0	X	- 0	0	9	63	18	Model
Thurst 526	100		7	0.0	-0.	:0:		0	0		×	1	
	432	62	8.6	Knife	6.0	63.0		. 0	0	9	63	. 6	20%
Float #1 1933	100		-	0.0	0	0		.0	0		1		F
Floor 40	520	88	10.4	Knife	6.0	63.0		.0	0		63	9	SPA
Fluor 39 1074	593	73	119	Knife	6.0	63.0		.0	0	9	63	9	5108
Hoor 38	999	73	13.3	Knife	0.0	63.0		.0	0	9	63	ió	45%
Floor 37 1298	739	23	14.8	Knife	8.0	63.0	Ý	0	0	9	63	89	4196
		74	16.3	Knife	6.0	63.0	Š	0.	.0	9	63	4	17%
Fluor 35 1522	888	74	17.8	Knife	6.0	63.0	À	.0	0	9	63	. 19	Matrill
Floor 34	963	75	19.3	Knife	6.0	63.0		.0	0	. 9	63	.9.	3306
Floor 33 1746	1038	. 75	20.8	Knife	6.0	63.0	×	.0	0		63	u	2906
Floor 32	1113	.75	22.3	Knife	6.0	63.0	x	0	0	15	63	19	279W
Floor 31 1970	1103	75.	23,8	Knife	6,0	63.0		0	0	9	63	.9:	250%
Floor 30	1764	316	151	Knife	60	63.0			0	4	- 63	9	2 dole
Floor 29 2194	1340	16	788	Knife	6.0	83.0	X	0	0		63	- 180	7562
	1417	16	28.3	Kniře	6.0	63.0	ý	0	0	. 9	63	9	21%
Floor 27 2418	1496	80	29.9	Knite	0.9	63.0	£	0	0	10	63	io	2008
Hoor 26	1576	000	315	Knife	6,0	63.0	1	0	.0	9	63	. 9	190F
Floor 25 2642	1656	80	33.1	Knife	6.0	63.0		. 0	0	9	63	9	18%
Floor 24	1739	83	34.8	Knife	6.0	63.0	x	0	0	ıs	63	فا	17/16
Fluor 23 2866		95	36.6	Knife	14.0	210,0	x	.0.	0	34	210	14.	956
		16	38.4	Knife	8.0	63.0		0	0	9	83	lap .	1946
Floor 21 3170	2013	16	40.3	Knife	.09	63.0		0	0	9	63	9	19w
	Ĭ	7.5	41.8	Knife	60	63.0		0	0		63	9	1405
Floor 19 3372	2164	75	43.3	Knife	6,0	63.0		0	0	. 9	63	19	14%
	2233	70	44.7	Knife	6.0	63.0		0	0	9	63	9	130
Floor 17 355R		92	46.1	Knife	6.0	63.0	,	0	0	15	63	9	1396
-		69	47.4	Knife	6,0	63.0	2	0	0	up.	63	100	1306
Floor 15 3744		-69	488	Kaife	0'9	63.0	(-)	0	0	9	63	le.	120
-		70	50.2	Knife	0.0	63.0	X	0	0	. 9	63	و	1681
Floor 13 3930		20	51.6	Knifc	6.0	63.0		0	ō	9	63	0	1296
	2	70	53.0	Knife	6.0	630	X	0	0		23	9	1108
Roor 11 4116		70	54.4	Kuife	60	63.0		0	0	9	63	to	17%
		17	089	Knife	60	630	<	0	0		63	ig:	1798
Floor 9 4302	2876	7.7	515	Knife	6.0	63.0		0	0	œ	60	Œ	Total
		11	0.99	Kaife	6.0	000		0	0	9	63	iá.	1000
Hoor 7 4528		-121	56.6	Trust Clux	0	NC	Knite	6.0	63.0	NC	MC	MC	>100
		121	69.0	Hn	121.1	131.1	X	0.0	0.0	121	121	121	205%
Hoor 5 4911		.358	51.9	Truss Cruc	NC	NC	X	00	0.0	NC	WC	NC	>100
		124	484	Fitz (+Angle)	1211	171.1		0.0	00	121	121	121	245%
Floor 3 5236	3363	2882	67.3	Truss Onx	NC	NC	Knife	10.3	105.0	NC	WC	NC	>100
Floor 2	3769	406	75.4	Fin (+Angle)	200.4	207.6	4	0.0	0.0	200	200	200	26699

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