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)

KATHERINE LINDSAY PRINGLE
FRIEDMAN KAPLAN SEILER & ADELMAN LLP
Seven Times Square, 28th Floor
New York, New York 10036
(212) 833-1100

Attorneys for Defendants-Cross-Defendants-Cross-Claimants-Third-Party-Plaintiffs-Appellees

THOMAS J. MOLONEY
CLEARY GOTTLIEB STEEN & HAMILTON LLP
One Liberty Plaza
New York, New York 10006
(212) 225-2000

Attorneys for Defendants-Cross-Defendants-Cross-Claimants-Appellees

FRANKLIN M. SACHS
GREENBAUM, ROWE, SMITH & DAVIS LLP
Metro Corporate Campus One
P.O. 5600
Woodbridge, New Jersey 07095
(732) 549-5600

– and –

MARK L. ANTIN
GENNET, KALLMANN, ANTIN & ROBINSON, P.C.
Six Campus Drive
Parsippany, New Jersey 07054
(973) 285-1919

Attorneys for Plaintiffs-Appellants

(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

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

Defendants,

THE WTC PLAINTIFFS,

Plaintiff-Intervenor.

KENNETH G. SCHWARZ
COZEN O'CONNOR
45 Broadway Atrium, 16th Floor
New York, New York 10006
(212) 509-9400

Attorneys for Defendant-Cross-Defendant-Appellee

DAVID ABRAMOVITZ
ZETLIN & DE CHIARA LLP
801 Second Avenue
New York, New York 10017
(212) 682-6800

– and –

STEPHEN P. SCHRECKINGER
GOGICK, BYRNE & O'NEILL, LLP
11 Broadway, Suite 1560
New York, New York 10004
(212) 422-9424

Attorneys for Defendants-Cross-Defendants-Third-Party-Defendants-Appellees

---

TABLE OF CONTENTS

	Page
Docket Entries	JA-1
Complaint, Filed September 10, 2004	JA-179
Answer, Filed December 9, 2004 on Behalf of Cantor Seinuk Group, P.C. and Office of Irwin G. Cantor, P.C.	JA-222
Answer and Cross-Claims, Filed December 30, 2004 on Behalf of 7 World Trade Center Company, L.P. and Silverstein Properties, Inc.	JA-259
Defendant, Flack & Kurtz, Inc.'s Notice of Motion to Dismiss, Filed January 4, 2005	JA-285
Declaration of Matthew S. Quinn, Esq. in Support of Flack & Kurtz Inc.'s Motion to Dismiss, Filed January 5, 2005, with the Following Attached Exhibits	JA-287
Exhibit A: Federal Emergency Management Agency's World Trade Center Building Performance Study	JA-289
Exhibit B: Port Authority of New York and New Jersey's Permit to Occupy and Use	JA-307
Exhibit C: Plaintiffs' Notice of Claim, Dated September 8, 2004	JA-308
Exhibit D: Complaint, Filed September 10, 2004 (Omitted Here but Reproduced at p. JA-179)	JA-311
Exhibit E: Excerpts of C-10 and C-11 of the Lease Between 7 World Trade Company and Salomon, Inc., Dated November 23, 1988	JA-312
Exhibit F: Report of Frederick W. Mowrer, Ph.D., Dated June 7, 2004	JA-315
Order Granting in Part and Denying in Part Motion of Flack & Kurtz, Inc., Filed January 10, 2005	JA-319

Amended Complaint, Filed February 7, 2005 .... JA-322

WTC7 Ground Construction Defendants' Notice of Motion to Dismiss, Filed April 21, 2005 .... JA-378

Third Declaration of Beth D. Jacob, Esq. in Support of WTC 7 Ground Defendants' Motions to Dismiss Plaintiffs' Complaints, Dated April 20, 2005, with the Following Attached Exhibits...... JA-382

Exhibit A: Amended Complaint Filed in Aegis Insurance Services, Inc., et al. v. Port Authority of New York and New Jersey, et al., 02 Civ. 7188, Dated March 6, 2002..... JA-386

Exhibit B: Second Amended Complaint Filed in Certain Underwrites of Lloyds, London, et al. v. The Port Authority of New York and New Jersey, 02 Civ. 7328, Dated April 4, 2003 .... JA-404

Exhibit C: Complaint, Filed September 10, 2004 (Omitted Here but Reproduced at p. JA-179) JA-420

Exhibit D: Amended Complaint, Filed February 7, 2005 (Omitted Here but Reproduced at p. JA-322) JA-421

Exhibit E: Federal Emergency Management Agency ("FEMA") Report Entitled "World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations", Dated May 2002 . JA-422

Exhibit F: Metropolitan Transit Authority's ("MTA") 2002 Annual Report JA-549

Exhibit G: Excerpt from the National Institute of Standards and Technology's Federal Building and Fire Safety Investigation of the World Trade Center Disaster Report Entitled, "The Emergency Response Operations (Draft)" Dated September 2005 .... JA-551

Exhibit H: Article by Graham Rayman, et al., Entitled A Sequence of Terror Unfolds, NEWSDAY, Dated September 23, 2001 .... JA-557

Exhibit I: Affidavit of Service of the Summons and Amended Complaint, Interrogatories and Other Documents on Cosentini Associates, Inc. in Aegis Insurance Services, Inc., et al. v. 7 World Trade Center Co., L.P. et al. 04 Civ. 7272, Dated March 7, 2005 .... JA-566

Exhibit J: Order Granting in Part and Denying in Part Motion of Flack & Kurtz, Inc., Filed January 10, 2005 (Omitted Here but Reproduced at p. JA-319).................................... JA-567

Exhibit K: Notice Pursuant to C.P.L.R. § 214-d to Skidmore Owings and Merrill, L.L.P., Dated September 2, 2004 ..... JA-568

Exhibit L: Letter from Mark L. Antin, Esq. to Skidmore Owings and Merrill, L.L.P., Dated September 8, 2004 and the Notice Pursuant to C.P.L.R. § 214-d to Skidmore Owings and Merrill, L.L.P., Dated September 7, 2004 ..... JA-572

Exhibit M: Notice Pursuant to C.P.L.R. § 214-d to Syska & Hennessy Engineers, Dated September 7, 2004 .... JA-577

Exhibit N: Letter from Mark L. Antin, Esq. to Flack & Kurtz, Inc., Dated September 8, 2004 and the Notice Pursuant to C.P.L.R. § 214-d to Flack & Kurtz, Inc., Dated September 7, 2004.. JA-581

Notice of Motion in Support of the Plaintiffs' Motion for Reconsideration of the Court's Ruling Granting Motions to Dismiss Filed by Certain Design, Construction and Contractor Defendants, Filed January 27, 2006 .... JA-586

Memo Endorsed Notice of Motion in Support of the Plaintiffs' Motion for Reconsideration of the Court's Ruling Granting Motions to Dismiss Filed by Certain Design, Construction and Contractor Defendants, Filed January 31, 2006, Signed by Judge Alvin K. Hellerstein JA-588

Answer and Affirmative Defenses, Filed February 22, 2006, on Behalf of Citigroup Inc. and Citigroup Global Markets Holdings, Inc. .. JA-590

Answer to Amended Complaint, Filed March 15, 2006, on Behalf of 7 World Trade Company, L.P. and Silverstein Properties, Inc. JA-611

Amended Answer of 7 World Trade Company, L.P. and Silverstein Properties, Inc. with Cross-Claims and Third Party Claims, Filed June 9, 2006 .... JA-630

Plaintiffs' Notice of Motion for an Order, Based Upon Newly Discovered Evidence, Vacating the Court's January 12, 2006 Order Granting a Fed. R. Civ. P. 12(b)(6) Motion Dismissing Plaintiffs' Complaint Against Defendants Emery Roth & Son, P.C., Irwin G. Cantor, P.C., and Cantor Seinuk Group, P.C., Filed July 24, 2006 .... JA-659

Affidavit of Mr. Richard King in Support of Plaintiffs' Motion, Based Upon Newly Discovered Evidence, Vacating the Court's January 12, 2006 Order Granting a Fed. R. Civ. P. 12(b)(6) Motion Dismissing Plaintiffs' Complaint Against Defendants Emery Roth & Son, P.C., Irwin G. Cantor, P.C., and Cantor Seinuk Group, P.C., Filed July 24, 2006 .... JA-661

Affidavit of Franklin M. Sachs, Esq., in Support of Plaintiffs' Motion, Based Upon Newly Discovered Evidence, Vacating the Court's January 12, 2006 Order Granting a Fed. R. Civ. P. 12(b)(6) Motion Dismissing Plaintiffs' Complaint Against Defendants Emery Roth & Son, P.C., Irwin G. Cantor, P.C., and Cantor Seinuk Group, P.C., Filed July 24, 2006, with the Following Attached Exhibit JA-664

Exhibit A: Consent Agreement, Dated March 31, 1982 .... JA-670

Order, Filed July 27, 2006 .... JA-698

Order, Filed August 8, 2006 .... JA-700

Stipulation Withdrawing Plaintiffs' Motion to Vacate without Prejudice, Filed December 12, 2006 .... JA-702

Second Amended Complaint, Filed July 11, 2008 .... JA-704

Answer of 7 World Trade Company, L.P., Silverstein Properties, Inc., and Silverstein Development Corp. to the Second Amended Complaint, with Cross-Claims and Third-Party Claims, filed August 6, 2008 .... JA-764

Answer and Affirmative Defenses to Second Amended Complaint, Filed October 2, 2008, on Behalf of Citigroup Inc. and Citigroup Global Markets Holdings, Inc. JA-772

Notice of Motion by Defendants 7 World Trade Company, L.P., Silverstein Development Corp. and Silverstein Properties, Inc. for Summary Judgment Dismissing all Claims Against Them, Filed November 24, 2009 .... JA-774

Statement Pursuant to Local Civil Rule 56.1 in Support of Defendants' Motion for Summary Judgment Dismissing Plaintiffs' Claims Asserted Against Them, Filed November 24, 2009 .... JA-776

Declaration of Jeffrey R. Wang, Esq. in Support of the Motion for Summary Judgment, Filed November 24, 2009, with the Following Attached Exhibits JA-788

Exhibit A: Second Amended Complaint, Filed July 11, 2008 (Omitted Here but Reproduced at p. JA-704) JA-792

Exhibit B: Agreement of Lease Between The Port Authority and Con Edison, Dated May 29, 1968 .... JA-793

Exhibit C: Excerpts of the Agreement of Lease Between The Port Authority and 7 World Trade Company, L.P., Dated December 31, 1980.. JA-828

Exhibit D: Excerpts of the Transcript of the Deposition Testimony of Larry A. Silverstein, Dated June 24, 2009 .. JA-845

Exhibit E: Plaintiffs' Memorandum of Law in Opposition to Silverstein's Motion to Dismiss, Dated March 18, 2008 ... JA-853

Exhibit F: Consent Agreement Between Con Edison and 7 World Trade Company, L.P., Dated March 1982 .... JA-868

Exhibit G: Excerpts of the Transcript of the Deposition Testimony of Richard King, Dated July 17, 2007 .... JA-897

Exhibit H: Excerpts of the Transcript of the Deposition Testimony of Frank P. Cruthers, Dated May 12, 2009 ...... JA-907

Exhibit I: Excerpts of the Transcript of the Deposition Testimony of Tiernach Cassidy, Dated May 15, 2009 ...... JA-914

Exhibit J: Excerpts of the Transcript of the Deposition Testimony of Richard A. Rotanz, Dated May 14, 2009 .... JA-918

Exhibit K: Excerpts of the Transcript of the Deposition Testimony of Rudolf C. Weindler, Dated May 6, 2009..... JA-925

Exhibit L: Excerpts of the Transcript of the Deposition Testimony of Christopher Boyle, Dated June 1, 2009....... JA-931

Exhibit M: Excerpts of the Transcript of the Deposition Testimony of Peter E. Hayden, Dated May 20, 2009 .... JA-935

Exhibit N: Excerpts of the Transcript of the Deposition Testimony of Terrence Roche, Dated May 19, 2009......... JA-940

Exhibit O: Excerpts of the Transcript of the Deposition Testimony of John S. Spiech, Dated May 27, 2009........... JA-944

Exhibit P: Excerpts of the Transcript of the Deposition Testimony of Daniel A. Nigro, Dated June 11, 2009 .... JA-948

Exhibit Q: Plaintiffs' Post-Hearing Memorandum of Law, Dated April 24, 2009 .... JA-951

Exhibit R: Letter from Robert Selya, Esq. to William Kufeld, Esq., Dated February 16, 1982, Regarding Consent Agreement JA-988

Exhibit S: Letter from Robert Selya, Esq. to William Kufeld, Esq., Dated March 8, 1982, Regarding Consent Agreement JA-989

Exhibit T: Memorandum in Support of Con Edison's Right to Pursue Its Claims Against Port Authority for Breach of Lease and Negligence in Connection with the Defective Design and Construction of World Trade Center 7 Atop Con Edison's Substation, and to Award Prejudgment Interest on Count III at 9% Per Year, Dated June 15, 2009, in *Aegis Insurance Services, Inc., et al. v. Port Authority of New York and New Jersey, et al.*, 02 Civ. 7188 .... JA-992

Exhibit U: Excerpts of the Transcript of the Deposition Testimony of Michael Patti, Dated June 19, 2009 .... JA-1028

Exhibit V: Excerpts of the Transcript of the Deposition Testimony of Allan Paull, Dated April 6, 2009 .... JA-1031

Exhibit W: Declaration of Colin G. Bailey, with Exhibits, Dated June 12, 2009, in *Aegis Insurance Services, Inc., et al. v. Port Authority of New York and New Jersey, et al.*, 02 Civ. 7188 .... JA-1034

Exhibit X: Typical Floor Framing Plan for 7 World Trade Company, L.P. JA-1050

Exhibit Y: Excerpts of the Transcript of the Deposition Testimony of Silvian Marcus, Dated August 12, 2009 .... JA-1051

Exhibit Z: Satisfaction of Judgment, Filed September 10, 2009, in *Aegis Insurance Services, Inc., et al. v. Port Authority of New York and New Jersey, et al.*, 02 Civ. 7188 .... JA-1053

Exhibit AA: Letter from Richard King to John Yurtchuk, Dated April 17, 1981, Regarding Consent Agreement JA-1055

Exhibit BB: Draft of Consent Agreement, Dated April 13, 1981 JA-1058

Exhibit CC: Draft of Consent Agreement, Dated July 10, 1981 .. JA-1062

Exhibit DD: Draft of Consent Agreement, Dated January 15, 1982 .... JA-1068

Exhibit EE: Minutes of Meeting Dated January 18, 1982 .... JA-1074

Exhibit FF: Affidavit of Richard King in Support of Plaintiffs' Motion, Based Upon Newly Discovered Evidence, Vacating the Court's January 12, 2006 Order Granting a Fed. R. Civ. P. 12(b)(6) Motion Dismissing Plaintiffs' Complaint Against Defendants Emery Roth & Son, P.C., Irwin G. Cantor, P.C., and Cantor Seinuk Group, P.C., Dated July 20, 2006 .... JA-1076

Notice of Motion for Summary Judgment in Favor of Citigroup Inc. and Citigroup Global Markets Holdings, Inc., Filed November 25, 2009 .... JA-1079

Defendants Citigroup Inc. and Citigroup Global Markets Holding, Inc.'s Statement of Undisputed Facts, Filed November 25, 2009 JA-1081

Declaration of Kristen M. Santillo, Esq. in Support of Citigroup Inc. and Citigroup Global Markets Holdings Inc.'s Motion for Summary Judgment, Filed November 25, 2009, with the Following Attached Exhibits JA-1106

Exhibit 1: Agreement of Lease Between The Port Authority and Con Edison, Dated May 29, 1968 .... JA-1111

Exhibit 2: Excerpt of Agreement of Lease Between The Port Authority and 7 World Trade Company, L.P., Dated December 31, 1980........ JA-1153

Exhibit 3: Excerpt of Contract Between Salomon Inc. and Skidmore Owings & Merrill, Dated September 7, 1988.... JA-1162

Exhibit 4: Excerpt of Agreement of Lease Between Salomon Inc., and Silverstein, Dated November 23, 1988 .... JA-1169

Exhibit 5: Excerpt of Three Party Agreement Among Salomon Inc., Silverstein, and Port Authority, Dated November 23, 1988 .... JA-1211

Exhibit 6: Excerpt of Contract between Salomon Inc. and Morse/Diesel, Dated January 23, 1989 .... JA-1295

Exhibit 7: Tenant Alteration Application, Dated May 30, 1989.. JA-1307

Exhibit 8: Excerpt of Contract Between Salomon Inc. and Flack & Kurtz, Dated June 1, 1989 .... JA-1332

Exhibit 9: Permit to Occupy Issued by Port Authority, Dated October 5, 1994 .... JA-1340

Exhibit 10: Complaint, Filed September 18, 2002 .... JA-1342

Exhibit 11: Complaint, Filed September 10, 2004 (Omitted Here but Reproduced at p. JA-179) JA-1350

Exhibit 12: Excerpts of the Transcript of the Deposition Testimony of Cornelius Lynch, Dated September 27, 2007 .... JA-1351

Exhibit 13: Excerpts of the Transcript of the Deposition Testimony of Robert White, Dated November 7 and 8, 2007 .... JA-1354

Exhibit 14: Excerpts of the Transcript of the Deposition Testimony of Eugene Fasullo, Dated December 18, 2007 and April 11, 2008 .... JA-1357

Exhibit 15: Excerpts of the Transcript of the Deposition Testimony of Guy Tozzoli, Dated January 10, 2008 .... JA-1361

Exhibit 16: Excerpts of the Transcript of the Deposition Testimony of Carl Borsari, Dated February 22, 2008 .... JA-1366

Exhibit 17: Excerpts of the Transcript of the Deposition Testimony of Gerry Gaeta, Dated February 28, 2008 .... JA-1372

Exhibit 18: Excerpts of the Transcript of the Deposition Testimony of Varoujan Dovletian, Dated June 11 and 12, 2008 .... JA-1385

Exhibit 19: Second Amended Complaint in *Aegis Ins. Servs.. Inc. v. The Port Authority of N.Y. &N.J.*, 02 Civ. 7188, Dated July 10, 2008 .... JA-1403

Exhibit 20: Excerpts of the Transcript of the Deposition Testimony of Manuel Fernandez, Dated July 18, 2008 .... JA-1418

Exhibit 21: Excerpts of the Transcript of the Deposition Testimony of Joseph Ritorto, Dated August 7, 2008 .... JA-1429

Exhibit 22: Excerpts of the Transcript of the Deposition Testimony of John Glass, Dated November 11, 2008 .... JA-1435

Exhibit 23: Excerpts of the Transcript of the Deposition Testimony of Chi Chu, Dated November 20, 2008 .... JA-1441

Exhibit 24: Excerpts of the Transcript of the Deposition Testimony of Irwin G. Cantor, Dated January 27 and 28, 2009 .... JA-1452

Exhibit 25: Excerpts of the Transcript of the Deposition Testimony of David Cooper, Dated February 5, 2009....... JA-1462

Exhibit 26: Excerpts of the Transcript of the Deposition Testimony of Jesse Bhatia, Dated March 12, 2009........... JA-1482

Exhibit 27: Excerpts of the Transcript of the Deposition Testimony of Rudolf C. Weindler, Dated May 6, 2009..... JA-1486

Exhibit 28: Excerpts of the Transcript of the Deposition Testimony of Frank Cruthers, Dated May 12, 2009 .... JA-1491

Exhibit 29: Excerpts of the Transcript of the Deposition Testimony of Richard Rotanz, Dated May 14, 2009 .... JA-1495

Exhibit 30: Excerpts of the Transcript of the Deposition Testimony of Tiernach Cassidy, Dated May 15, 2009 ...... JA-1500

Exhibit 31: Excerpts of the Transcript of the Deposition Testimony of Terrence Roche, Dated May 19, 2009......... JA-1503

Exhibit 32: Excerpts of the Transcript of the Deposition Testimony of Peter Hayden, Dated May 20, 2009............. JA-1507

Exhibit 33: Excerpts of the Transcript of the Deposition Testimony of Allen S. Hay, Dated May 21, 2009.............. JA-1510

Exhibit 34: Excerpts of the Transcript of the Deposition Testimony of John Spiech, Dated May 27, 2009............... JA-1513

Exhibit 35: Excerpts of the Transcript of the Deposition Testimony of Christopher Boyle, Dated June 1, 2009 ...... JA-1518

Exhibit 36: Excerpts of the Transcript of the Deposition Testimony of Anthony Varriale, Dated June 9, 2009 .... JA-1522

Exhibit 37: Excerpts of the Transcript of the Deposition Testimony of Daniel Nigro, Dated June 11, 2009 .... JA-1525

Exhibit 38: Excerpts of the Transcript of the Deposition Testimony of Philip Ruvolo, Dated June 12, 2009 .... JA-1532

Exhibit 39: Declaration of Frederick W. Mowrer, Dated June 12, 2009 .... JA-1535

Exhibit 40: Declaration of Guy Nordenson, Dated June 15, 2009 .... JA-1540

Exhibit 41: Memorandum in Support of Con Edison's Right to Pursue Its Claims Against Port Authority for Breach of Lease and Negligence in Connection with the Defective Design and Construction of World Trade Center 7 Atop Con Edison's Substation, and to Award Prejudgment Interest on Count III at 9% Per Year, Dated June 15, 2009, in *Aegis Insurance Services, Inc., et al. v. Port Authority of New York and New Jersey, et al.*, 02 Civ. 7188 (Omitted Here but Reproduced at p. JA-992) JA-1543

Exhibit 42: Excerpts of the Transcript of the Deposition Testimony of Anthony Zeolla, Dated June 18, 2009 .... JA-1544

Exhibit 43: Excerpts of the Transcript of the Deposition Testimony of Joseph Kelly, Dated June 19, 2009.............. JA-1548

Exhibit 44: Excerpts of the Transcript of the Deposition Testimony of Paul Ferro, Dated June 23, 2009.................. JA-1551

Exhibit 45: Excerpts of the Transcript of the Deposition Testimony of Larry Silverstein, Dated June 24, 2009 .... JA-1554

Exhibit 46: Excerpts of the Transcript of the Deposition Testimony of Bruce Gavioli, Dated August 5, 2009 .... JA-1566

Exhibit 47: Excerpts of the Transcript of the Deposition Testimony of Silvian Marcus, Dated August 12, 2009...... JA-1575

Exhibit 48: Letter from Con Edison to Defendants Regarding Damages, Dated October 1, 2009...................................... JA-1579

Exhibit 49: Excerpts of the Transcript of the Deposition Testimony of Anthony Zizolfo, Dated October 10, 2009.. JA-1596

Exhibit 50: Excerpts of the Transcript of the Deposition Testimony of Robert Mudzinski, Dated October 14, 2009 JA-1602

Exhibit 51: Excerpts of the Transcript of the Deposition Testimony of Jeffrey Smilow, Dated October 22, 2009 .... JA-1605

Exhibit 52: Excerpts of the Transcript of the Deposition Testimony of Richard Peck, Dated November 9, 2009 ..... JA-1614

Exhibit 53: Guidelines for Professional Engineering Practice in New York State, November 19, 2009 (last viewed) JA-1617

Plaintiffs' Notice of Cross-Motion to Stay Determination of Silverstein Defendants' Summary Judgment Motion as to Count XVII Pending Resolution of Appeal in 02-CV-7188, Dated February 1, 2010 .... JA-1621

Declaration of Jemi M. Goulian, Esq. in Support of Plaintiffs' Opposition to Defendants' Motions for Summary Judgment and in Support of Plaintiffs' Motion to Stay, Filed February 1, 2010, with the Following Attached Exhibits... JA-1623

Exhibit 1: Excerpts of the Transcript of the Deposition Testimony of Guy F. Tozzoli, Dated January 10, 2008..... JA-1634

Exhibit 2: Agreement of Lease Between The Port Authority and Con Edison, Dated May 29, 1968 (Omitted Here but Reproduced at p. JA-793).. JA-1639

Exhibit 3: Excerpts of the Transcript of the Deposition Testimony of Richard L. Groccia, Dated November 29, 2007 .... JA-1640

Exhibit 4: Excerpts of the Transcript of the Deposition Testimony of Cornelius Lynch, Dated September 26, 2007 .... JA-1644

Exhibit 5: Deposition Exhibit Lynch-1, which was Marked as an Exhibit at Cornelius Lynch's Deposition, Dated September 26, 2007 .... JA-1649

Exhibit 6: Deposition Exhibit Lynch-2, which was Marked as an Exhibit at Cornelius Lynch's Deposition, Dated September 26, 2007 .... JA-1652

Exhibit 7: Excerpts of Deposition Exhibit Bach-1 which was Marked as an Exhibit at Arthur Bach's Deposition, Dated January 15, 2008........ JA-1655

Exhibit 8: Excerpts of the Transcript of the Deposition Testimony of Varoujan Dovletian, Dated June 11, 2008... JA-1680

Exhibit 9: Excerpts of the Transcript of the Deposition Testimony of Larry A. Silverstein, Dated June 24, 2009 .. JA-1686

Exhibit 10: Deposition Exhibit King-8, which was Marked as an Exhibit at Richard King's Deposition, Dated July 17, 2007 .... JA-1698

Exhibit 11: Deposition Exhibit Ritorto-1, which was Marked as an Exhibit at Joseph Ritorto's Deposition, Dated August 7, 2008........ JA-1714

Exhibit 12: Deposition Exhibit Jupiter-1, which was Marked as an Exhibit at Harold Jupiter's Deposition, Dated April 25, 2008 .... JA-1719

Exhibit 13: Deposition Exhibit Silverstein-4, which was Marked as an Exhibit at Larry A. Silverstein's Deposition, Dated June 24, 2009 .... JA-1723

Exhibit 14: Deposition Exhibit Silverstein-6, which was Marked as an Exhibit at Larry A. Silverstein's Deposition, Dated June 24, 2009 .... JA-1725

Exhibit 15: Document Entitled "Building Description: 7 World Trade Center," Bearing Bates Numbers 7WTC-0007320 . JA-1739

Exhibit 16: Document Entitled "Significant Terrorist Incidents, 1961-2003: A Brief Chronology, *U.S. Department of State, March 2004*".. JA-1748

Exhibit 17: Deposition Exhibit Silverstein-1, which was Marked as an Exhibit at Larry A. Silverstein's Deposition, Dated June 24, 2009 (Confidential) (Omitted here but printed at p. CJA-1).. JA-1764

Exhibit 18: News Article Published by *The New York Times* on January 14, 1980, Entitled "Bomb Damages Offices of Soviet Airlines on Fifth Ave.".. JA-1765

Exhibit 19: News Article Published by *The New York Times* on October 13, 1980, Entitled "Blast Injures Four Near a U.N. Mission".. JA-1767

Exhibit 20: News Article Published by *The New York Times* on January 24, 1981, Entitled "Bomb Shatters Windows in Courthouse Downtown"... JA-1769

Exhibit 21: News Article Published by *The New York Times* on March 1 and 2, 1982, Entitled "At Least 4 Bombs Rock Wall Street; Exchanges are Hit", and "Wall Street Bombing Laid to Resurgence of F.A.L.N." JA-1770

Exhibit 22: News Article Published by *The New York Times* on September 20, 1982, Entitled "Bank on Park Avenue is Bombed"... JA-1774

Exhibit 23: News Article Published by *The New York Times* on January 2 and 4, 1983, Entitled "F.A.L.N. Puerto Rico Terrorists Suspected in New Year Bombings"', and "Koch Triples Reward Offered in New Year Bombings Case"... JA-1775

Exhibit 24: News Article Published by *The New York Times* on January 30, 1983, Entitled "Federal Office Building on Staten Island Damaged by a Bomb" JA-1779

Exhibit 25: News Article Published by *The New York Times* on February 24, 1985, Entitled "2 Women Seen at Site of Blast" JA-1780

Exhibit 26: News Article Published by *The New York Times* on July 11, 1986, Entitled "A Realty Gambler's Big Coup" .. JA-1781

Exhibit 27: News Article Published by *The New York Times* on November 21, 1986, Entitled "Drexel 'Junk Bond' Wizard" JA-1784

Exhibit 28: Excerpts of Deposition Exhibit Schnabel-7, which was Marked as an Exhibit at Donald Schnabel's Deposition, Dated February 12, 2008 .... JA-1787

Exhibit 29: Excerpts of Deposition Exhibit Geata-2, which was Marked as an Exhibit at Gerard Gaeta's Deposition, Dated February 28, 2008 .... JA-1839

Exhibit 30: Deposition Exhibit Schnabel-5, which was Marked as an Exhibit at Donald Schnabel's Deposition, Dated February 12, 2008 .... JA-1849

Exhibit 31: Deposition Exhibit Borsari-15, which was Marked as an Exhibit at Carl Borsari's Deposition, Dated February 22, 2008 .... JA-1852

Exhibit 32: Deposition Exhibit Glass-8, which was Marked as an Exhibit at John Glass' Deposition, Dated November 11, 2008 .... JA-1855

Exhibit 33: Deposition Exhibit Glass-4, which was Marked as an Exhibit at John Glass' Deposition, Dated November 11, 2008 .... JA-1858

Exhibit 34: Deposition Exhibit Glass-1, which was Marked as an Exhibit at John Glass' Deposition, Dated November 11, 2008 .... JA-1860

Exhibit 35: Excerpts of the Transcript of the Deposition Testimony of Chi Chu, Dated November 20, 2008 .... JA-1861

Exhibit 36: Deposition Exhibit Borsari-10, which was Marked as an Exhibit at Carl Borsari's Deposition, Dated February 22, 2008 .... JA-1864

Exhibit 37: Deposition Exhibit Chu-2, which was Marked as an Exhibit at Chi Chu's Deposition, Dated November 20, 2008 .... JA-1865

Exhibit 38: News Article Published by The New York Times on November 13, 1997, Entitled "The Trade Center Verdict: The Overview; 'Mastermind' and Driver Found Guilty in 1993 Plot to Blow Up Trade Center" JA-1879

Exhibit 39: Deposition Exhibit Ritorto-9, which was Marked as an Exhibit at Joseph Ritorto's Deposition, Dated August 7, 2008 .... JA-1883

Exhibit 40: Deposition Exhibit Weems-4, which was Marked as an Exhibit at Walter Weems' Deposition, Dated December 13, 2007 .... JA-1886

Exhibit 41: Excerpts of the Transcript of the Deposition Testimony of Stephen G. Nathan, Dated July 30, 2007 .... JA-1887

Exhibit 42: Copy of CD Containing Various Videos of the Collapse of 7 World Trade Center Taken from Several Angles JA-1891

Exhibit 43: Excerpts of the Transcript of the Deposition Testimony of Frank P. Cruthers, Dated May 12, 2009 .... JA-1892

Exhibit 44: Excerpts of the Transcript of the Deposition Testimony of Tiernach Cassidy, Dated May 15, 2009 .... JA-1896

Exhibit 45: Excerpts of the Transcript of the Deposition Testimony of Richard A. Rotanz, Dated May 14, 2009 .... JA-1900

Exhibit 46: Excerpts of the Transcript of the Deposition Testimony of Daniel A. Nigro, Dated June 11, 2009 .... JA-1908

Exhibit 47: Excerpts of the Transcript of the Deposition Testimony of Michael Patti, Dated June 19, 2009 .... JA-1914

Exhibit 48: Excerpts of the Transcript of the Deposition Testimony of Michael Catalano, Dated January 30, 2008. JA-1917

Exhibit 49: Excerpts of the New York City Building Code Revision, Dated October 4, 2004, as Well as the List of Amendments JA-1921

Exhibit 50: Excerpts of the Transcript of the Deposition Testimony of David Cooper, Dated February 5, 2009....... JA-1954

Exhibit 51: Deposition Exhibit White-7, which was Marked as an Exhibit at Robert C. White's Deposition, Dated November 8, 2007 .... JA-1964

Exhibit 52: Excerpts of the Transcript of the Deposition Testimony of Manuel Fernandez, Dated July 18, 2008 ..... JA-1968

Exhibit 53: Deposition Exhibit White-5, which was Marked as an Exhibit at Robert C. White's Deposition, Dated November 8, 2007 .... JA-1971

Exhibit 54: Excerpts of the Transcript of the Deposition Testimony of Colvin Cox, Dated July 31, 2007 .... JA-1975

Exhibit 55: Excerpts of the Transcript of the Deposition Testimony of Terrence Roche, Dated May 19, 2009......... JA-1978

Exhibit 56: Excerpts of the Transcript of the Deposition Testimony of Daniel Delargy, Dated June 17, 2009.......... JA-1981

Exhibit 57: Excerpts of the Transcript of the Deposition Testimony of Mark A. Giannini, Dated June 29, 2009...... JA-1985

Exhibit 58: Excerpts of the Transcript of the Deposition Testimony of Peter E. Hayden, Dated May 20, 2009 .... JA-1988

Exhibit 59: Excerpts of the Transcript of the Deposition Testimony of Joseph Meola, Dated June 24, 2009 .... JA-1991

Exhibit 60: Excerpts of the Transcript of the Deposition Testimony of Anthony Varriale, Dated June 9, 2009 .... JA-1995

Exhibit 61: Excerpts of the Transcript of the Deposition Testimony of Peter Joseph Mulroy, Dated August 27, 2007 .... JA-1999

Exhibit 62: Deposition Exhibit Delargy-3, which was Marked as an Exhibit at Daniel Delargy's Deposition, Dated June 17, 2000 .... JA-2002

Exhibit 63: Deposition Exhibit Cruthers-5, which was Marked as an Exhibit at Frank P. Cruthers' Deposition, Dated May 12, 2009 .... JA-2003

Exhibit 64: Excerpts of the Transcript of the Deposition Testimony of Irwin G. Cantor, Dated January 27, 2009 .... JA-2004

Exhibit 65: Deposition Exhibit Borsari-2, which was Marked as an Exhibit at Carl Borsari's Deposition, Dated February 22, 2008 .... JA-2011

Exhibit 66: Deposition Exhibit Gilsanz-7, which was Marked as an Exhibit at Ramon Gilsanz's Deposition, Dated May 28, 2009 .... JA-2013

Exhibit 67: Deposition Exhibit Gilsanz-6, which was Marked as an Exhibit at Ramon Gilsanz's Deposition, Dated May 28, 2009 .... JA-2020

Exhibit 68: Second Amended Complaint, Filed July 11, 2008 (Omitted Here but Reproduced at p. JA-704) JA-2026

Exhibit 69: Answer of 7 World Trade Company, L.P., Silverstein Properties, Inc., and Silverstein Development Corp. to the Second Amended Complaint, with Cross-Claims and Third-Party Claims, filed August 6, 2008 (Omitted Here but Reproduced at p. JA-764) JA-2027

Exhibit 70: Answer and Affirmative Defenses to Second Amended Complaint, Filed October 2, 2008, on Behalf of Citigroup Inc. and Citigroup Global Markets Holdings, Inc. (Omitted Here but Reproduced at p. JA-772) JA-2028

Exhibit 71: Affidavit of Barbara A. Carey, Dated October 27, 2008 .... JA-2029

Exhibit 72: Excerpts of Transcript of Proceedings, Dated February 23, 2009 in 04-CV-7272 .... JA-2031

Exhibit 73: Letter from John Yurtchuk to Richard King, Dated March 6, 1981, Bearing Bates Number CANTOR2003105........ JA-2035

Exhibit 74: Deposition Exhibit Rubino-7, which was Marked as an Exhibit at Anthony Rubino's Deposition, Dated March 27, 2008........ JA-2036

Exhibit 75: Excerpts of the Transcript of the Deposition Testimony of Michael E. Smith, Dated January 15, 2009 . JA-2039

Exhibit 76: Deposition Exhibit Cooper-17, which was Marked as an Exhibit at David Cooper's Deposition, Dated February 5, 2009........ JA-2042

Exhibit 77: Excerpts of the Transcript of the Deposition Testimony of Richard P. Fogarty, Dated October 26, 2009 .... JA-2050

Exhibit 78: U.S. Court of Appeals for the Second Circuit, General Docket Referencing the Current Appellate Scheduling Order........ JA-2054

Exhibit 79: Deposition Exhibit King-22, which was Marked as an Exhibit at Richard King's Deposition, Dated July 17, 2007 .... JA-2063

Exhibit 80: Notice of Appeal, Filed August 21, 2009 in 02-CV-7188 .... JA-2064

Exhibit 81: Letter from William M. Kufeld to Robert P. Selya, Dated February 9, 1982, Bearing Bates Number SPI025872 .... JA-2065

Exhibit 82: Transcript of Proceedings, Dated July 7, 2008........ JA-2066

Second Declaration of Colin G. Bailey, Filed February 1, 2010, with the Following Attached Exhibits: JA-2084

Exhibit A: Colin Gareth Bailey's Curriculum Vitae JA-2088

Exhibit B: Photographs Showing Fire Protection on a Different Building JA-2099

Exhibit C: Photograph of SFRM Being Applied JA-2108

Exhibit D: Photograph of Flutes Filled with SFRM JA-2109

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

Second Declaration of Kenneth Elovitz, Filed February 1, 2010, with the Following Attached Exhibit JA-2123

Exhibit A: Kenneth M. Elovitz's Curriculum Vitae JA-2129

Second Declaration of Frederick W. Mowrer, Filed February 1, 2010, with the Following Attached Exhibits JA-2134

Exhibit A: Frederick W. Mowrer's Curriculum Vitae JA-2140

Exhibit B: Photograph of the East Side of 7 World Trade Center JA-2150

Exhibit C: Morse Diesel Photographs Depicting Unfilled Flute Cavities JA-2151

Second Declaration of Guy Nordenson, Filed February 1, 2010, with the Following Attached Exhibit JA-2155

Exhibit A: Guy J.P. Nordenson's Curriculum Vitae JA-2158

Second Declaration of Joseph M. Sorge, Filed February 1, 2010, with the Following Attached Exhibit JA-2168

Exhibit A: Joseph M. Sorge's Curriculum Vitae JA-2170

Declaration of Jose L. Torero, Filed February 1, 2010, with the Following Attached Exhibit JA-2175

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

Plaintiffs' Response to Defendants Citigroup Inc. and Citigroup Global Markets Holing Inc.'s Local Civil Rule 56.1 Statement of Undisputed Facts, Filed February 1, 2010 .... JA-2220

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

Exhibit A: Underground Storage Tank Closure Report, Dated September 9, 2002 .... JA-2252

Exhibit B: Closure Report for Transformer Oil Release at 7 World Trade Center, Dated January 2004.......................... JA-2258

Exhibit C: Excerpts of the Transcript of the Deposition Testimony of Anthony Varriale, Dated June 9, 2009 .... JA-2264

Exhibit D: Excerpts of the Transcript of the Deposition Testimony of Phillip C. Ruvolo, Dated June 12, 2009 ...... JA-2267

Exhibit E: Excerpts of the Transcript of the Deposition Testimony of Peter Joseph Mulroy, Dated August 27, 2007 .... JA-2269

Exhibit F: Excerpts of the Transcript of the Deposition Testimony of Colvin Cox, Dated July 31, 2007 .... JA-2271

Exhibit G: Excerpts of the Transcript of the Deposition Testimony of John S. Spiech, Dated May 27, 2009........... JA-2273

Exhibit H: Excerpts of the Transcript of the Deposition Testimony of Allen S. Hay, Dated May 21, 2009.............. JA-2276

Exhibit I: Excerpts of the Transcript of the Deposition Testimony of Rudolf C. Weindler, Dated May 6, 2009..... JA-2278

Exhibit J: Excerpts of the Transcript of the Deposition Testimony of Anthony Zeolla, Dated June 18, 2009 .... JA-2282

Exhibit K: Excerpts of the Transcript of the Deposition Testimony of Christopher Boyle, Dated June 1, 2009 .... JA-2284

Exhibit L: Excerpts of the Transcript of the Deposition Testimony of Barry "Butch" Brandes, Dated July 14, 2009 .... JA-2286

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

Exhibit O: Excerpts of the Transcript of the Deposition Testimony of Michael Patti, Dated June 19, 2009 .... JA-2292

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

Exhibit R: *The New York Times* Article: "Briefcase Found at Bombing Site," Dated January 27, 1975 .... JA-2302

Exhibit S: Excerpts of the Transcript of the Deposition Testimony of Joseph Ritorto, Dated August 7, 2008 .... JA-2303

Exhibit T: Excerpts of Report "Empire State Development Corporation Attestation Engagement for Utility Restoration and Infrastructure Rebuilding Program – Consolidated Edison Company of New York, Inc. Category Two Filing," Dated February 22, 2007 .... JA-2306

Declaration of Craig L. Beyler, Filed March 19, 2010, with the Following Attached Exhibits JA-2312

Exhibit A: Craig L. Beyler's Curriculum Vitae JA-2323

Exhibit B: Selected Publication List of Craig L. Beyler, Ph.D. ... JA-2326

Exhibit C: Photographs of Debris Strikes Causing Multiple Ignitions in Locations Throughout the Day JA-2330

Exhibit D: Photographs of Filled Flutes JA-2338

Exhibit E: Photographs of Filled Flutes JA-2340

Citigroup Inc. and Citigroup Global Markets Holdings Inc.'s Reply Statement of Undisputed Material Facts, Filed March 19, 2010 . JA-2344

Declaration of Chester T. Vogel, Filed March 19, 2010, with the Following Attached Exhibits JA-2404

Exhibit A: Chester T. Vogel's Curriculum Vitae JA-2412

Exhibit B: Excerpts of Fuel Oil Piping and Storage Manual JA-2413

Exhibit C: Excerpts of New York City Building Code Title 27-Subchapter 3 Occupancy and Construction Classification JA-2423

Exhibit D: Excerpts of Electric Power Generation Transmission Distribution Manual JA-2433

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 the Following Attached Exhibits JA-2435

Exhibit 1: Excerpts of Agreement of Lease Between Salomon Inc. and Silverstein, Dated November 23, 1988 .... JA-2438

Exhibit 2: Three Party Agreement Among Salomon Inc., Silverstein, and Port Authority, Dated November 23, 1988 .... JA-2458

Exhibit 3: GZA GeoEnvironmental Inc., Underground Storage Tank Closure Report, Dated September 9, 2002 .... JA-2466

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

Exhibit 5: Excerpts of the Transcript of the Deposition Testimony of Varoujan Dovletian, Dated June 12, 2008 (Further Supplementing Santillo Declaration Exhibit 18) . JA-2493

Exhibit 6: Excerpts of the Transcript of the Deposition Testimony of David Cooper, Dated February 5, 2009 (Further Supplementing Santillo Declaration Exhibit 25) . JA-2497

Exhibit 7: Transcript of Oral Argument and Status Conference in *Aegis Ins. Servs., Inc. v. Port Auth. of N.Y. & N.J.*, 04-CV-7272, Dated February 23, 2009 .... JA-2500

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

Exhibit 9: Excerpts of the Transcript of the Deposition Testimony of Richard Rotanz, Dated May 14, 2009 (Further Supplementing Santillo Declaration Exhibit 29) . JA-2505

Exhibit 10: Excerpts of the Transcript of the Deposition Testimony of Tiernach Cassidy, Dated May 15, 2009 (Further Supplementing Santillo Declaration Exhibit 30) . JA-2509

Exhibit 11: Excerpts of the Transcript of the Deposition Testimony of Terrence Roche, Dated May 19, 2009 (Further Supplementing Santillo Declaration Exhibit 31) . JA-2513

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

Exhibit 16: Excerpts of the Transcript of the Deposition Testimony of Daniel Delargy, Dated June 17, 2009 .... JA-2531

Exhibit 17: Excerpts of the Transcript of the Deposition Testimony of Joseph Meola, Dated June 24, 2009 .... JA-2535

Exhibit 18: Excerpts of the Transcript of the Deposition Testimony of Mark Giannini, Dated June 29, 2009........... JA-2539

Exhibit 19: Excerpts of the Transcript of the Deposition Testimony of Barry Brandes, Dated July 14, 2009............ JA-2543

Exhibit 20: Con Edison September 11, 2001 Restoration Spreadsheet, Dated August 31, 2009 .... JA-2548

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

1. 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^{th} & 6^{th} 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 5th and 10th, 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

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

I, Guy Nordenson, declare:

1. 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.

5. 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.[1]

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

[1] 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.

Guy Nordenson and Associates

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

Professional Experience

1997-date *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 Raimund 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 Maltzan 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 Canopies, 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 Arquitectos 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 HQ, New York NY (Pei Cobb Freed & Partners, with Yolles Partnership) 2004
59 East 59 Theater, New York NY (UrED/Leo Modrcin Architect) 2004
Bridges Center, Memphis TN (Building Studio with Coleman Coker Architects) 2004

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 & 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, Kauai 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
Bellevue 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, (Pei 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 Sekkei and Shimizu - 2nd place) 1997

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 AIA 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 seismic 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

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 Daily 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

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
AIA 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
AIA 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 Biennale, 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 – 8 Mostra Internationale de Architettura-la Biennale de Venezia, Venice ITALY 2002
Art on the Beach, with Jody Culkin and Uwe Mengel, Creative Time, New York NY 1988

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 Hazard 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' *Perspecta 31: Reading Structures*, Yale Architecture Journal, New Haven CT 2000
'4 Experimental Projects' *Dialogue*, Taipei TAIWAN 2000
'Seismic 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*, Daidalos 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

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, Soil 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 Nat 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-Buildings 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 *Jrnl 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, 'AIA 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 2003
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

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 Iovine, '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

Current

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
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

Past (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
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

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

Katherine L. Pringle, Esq.
Friedman, Kaplan, Seiler & Adelman LLP
1633 Broadway, 46th Floor
New York, NY 10019

Christopher P. Moore, Esq.
Cleary, Gottlieb, Steen & Hamilton LLP
One Liberty Plaza
New York, NY 10006

Guy Nordenson and Associates

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

By

Guy Nordenson PE, SE

Structural Engineers LLP

225 Varick Street 6th Flr
New York NY 10014 USA
Tel 212 766 9119
Fax 212 766 9016
www.nordenson.com

TABLE OF CONTENTS

		Page
1.0	**EXECUTIVE SUMMARY**	1
2.0	**INTRODUCTION**	**5**
2.1	Description of Structure	5
2.2	Description of Collapse on 11 September 2001	10
2.3	Description of Global Collapse Analysis Approach	11
3.0	**ANALYSIS BASIS**	**14**
3.1	Document Review and Use	14
3.2	Loading Assumptions	15
3.3	Consideration of Debris and Fire Damage	17
3.4	Material Strength and Structural Capacities	19
3.5	Additional Analysis Details	20
4.0	**ASSESSMENT OF STRUCTURAL VULNERABILITY**	**24**
4.1	General Structural Vulnerabilities	24
4.2	Lateral Bracing Code Violation	28
4.3	Summary of Structural Vulnerabilities	64
5.0	**PROBABLE GLOBAL COLLAPSE MECHANISM**	**65**
5.1	Summary of Visual Evidence of Collapse	65
5.2	Basis of Staged Deconstruction	67
5.3	Summary of Probable Global Collapse Mechanism	68
5.4	Probable Collapse Sequence Stage 1 Analysis Details	78
5.5	Probable Collapse Sequence Stage 2 Analysis Details	82
5.6	Probable Collapse Sequence Stage 3 Analysis Details	92
5.7	Probable Collapse Sequence Stage 4 Analysis Details	96
5.8	Probable Collapse Sequence Stage 5 Analysis Details	103
5.9	Probable Collapse Sequence Stage 6 Analysis Details	111
6.0	**SUMMARY OF FINDINGS**	**114**
7.0	**REFERENCES**	**116**

APPENDICES

A.0 APPENDIX A – WTC7 Knife Connection Study by Anthony Ingraffea PE PhD

B.0 APPENDIX B – Floor Collapse Analysis Report

C.0 APPENDIX C – Interior Column Stability Analysis Report

D.0 APPENDIX D – Floor Diaphragm Rupture Analysis Report

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 probable[1] 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.

---

[1] When used in this report as part of an opinion, the word "probable" means "to a reasonable degree of scientific probability"

Stage 1

Stage 2

Stage 3

Stage 4

Stage 5

Stage 6

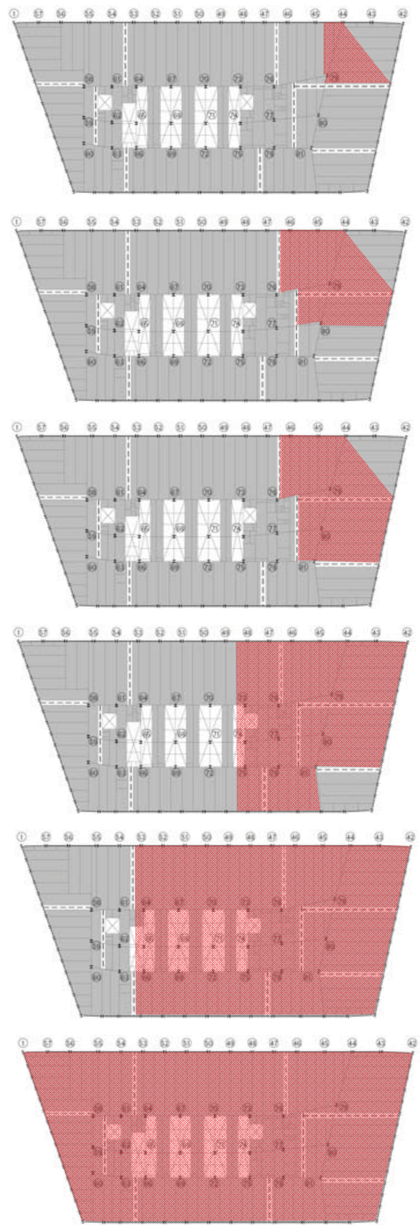


Figure 1.1 Stages of global collapse

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).

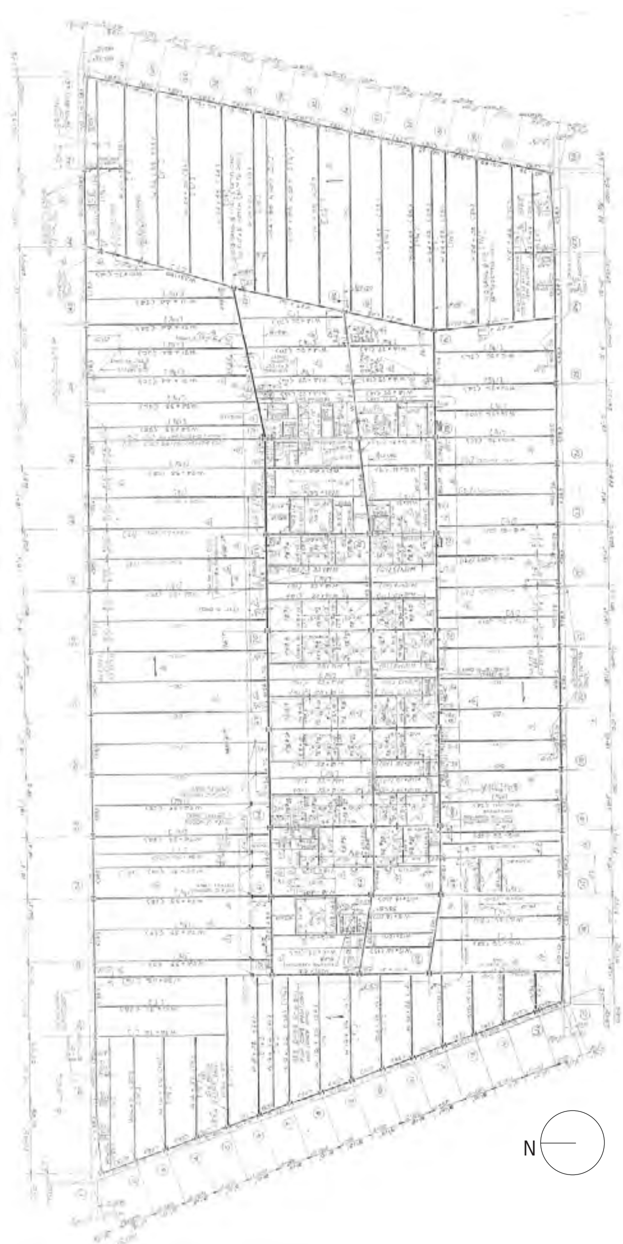


Figure 2.1 WTC7 typical floor framing plan from Cantor structural drawings (TISHMAN014724)

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.

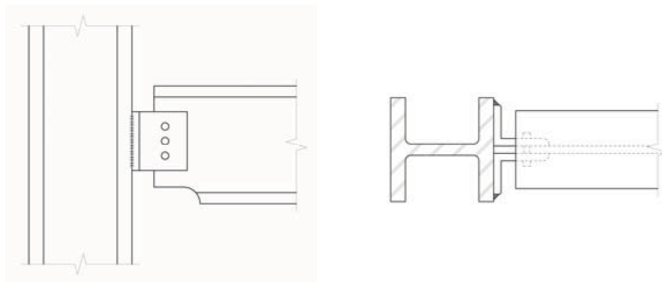
Northeast corner framing as-designed
(floor beams framing in two directions)

Alternate northeast corner floor framing
(floor beams framing in one direction)

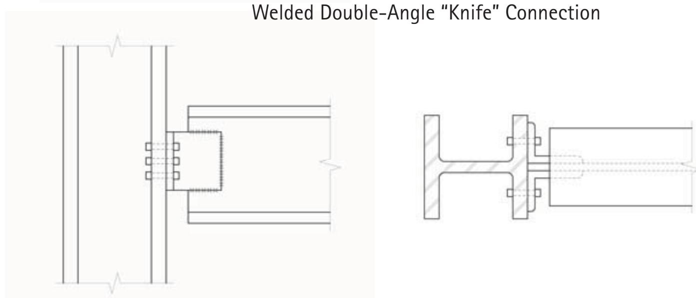
Figure 2.2 Configuration of typical floor framing at northeast corner

2.1.3 Connection Details

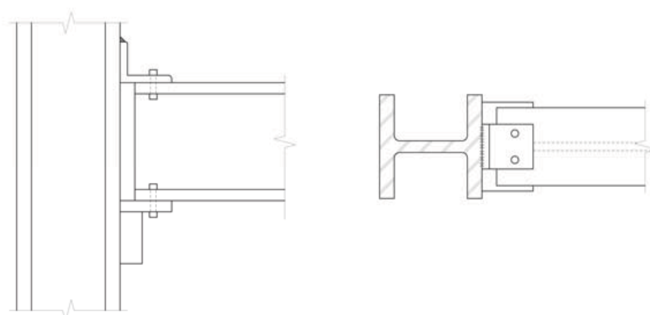
According to the steel shop drawings, the primary connection details between the girders and columns fall into several classifications: double-angle “knife” type connections (bolted to girder and welded to column), double-angle “header” type connections (welded to girder and bolted to column), and seated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



Welded Double-Angle "Knife" Connection



Bolted Double-Angle "Header" Connection



Seated 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).

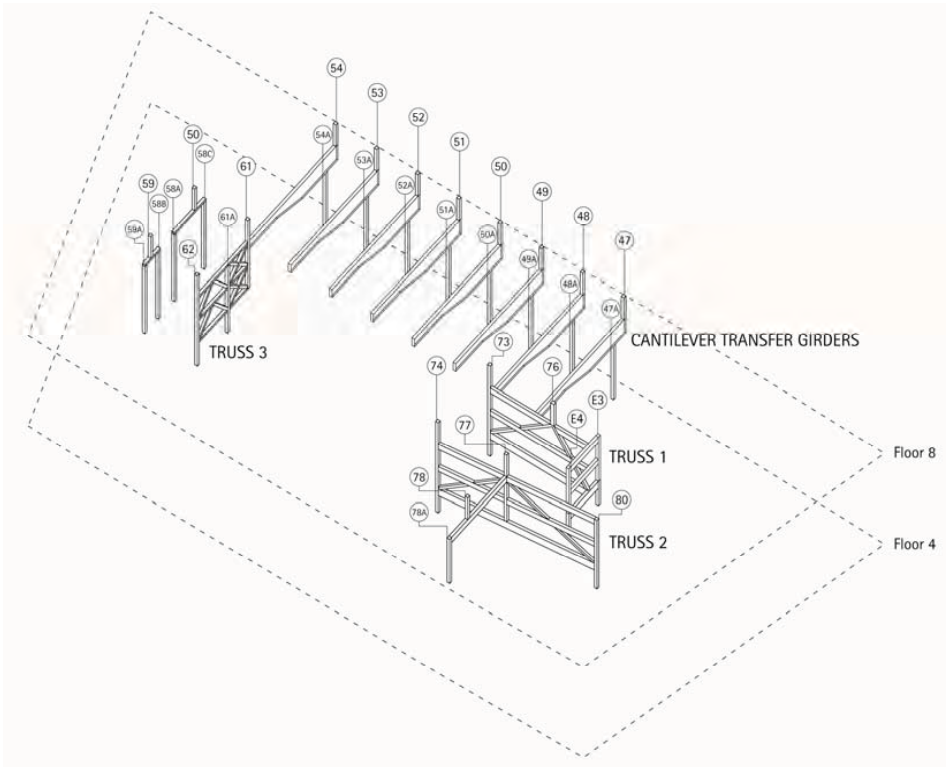


Figure 2.4 Transfer trusses and girders located between Floors 5 and 7

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.

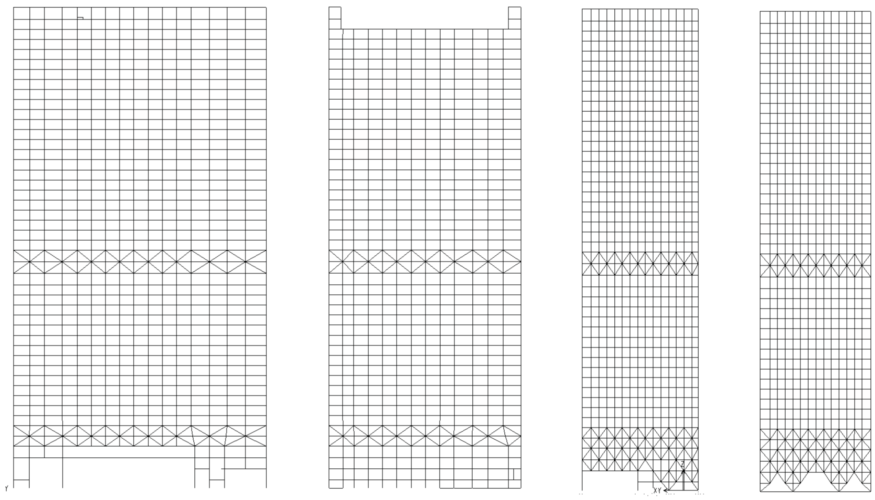


Figure 2.5 Structural elevations of WTC7

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).

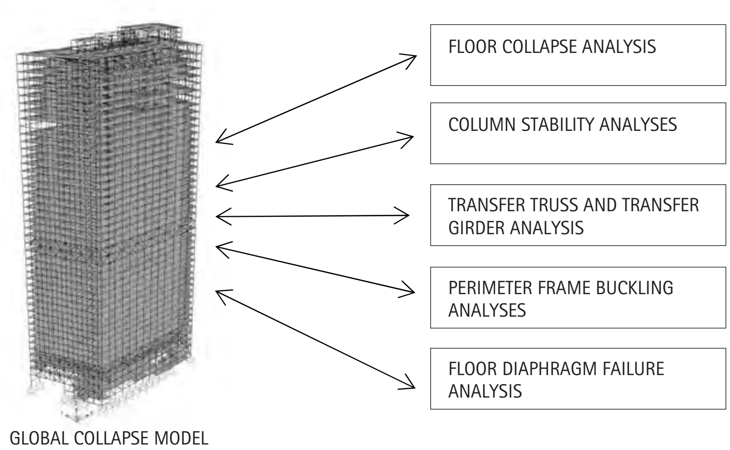


Figure 2.6 Global collapse model and sub-model interface

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).

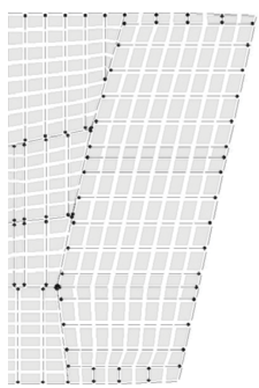


Figure 2.7 Floor slab element meshing in partial view of global model

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 SLAB DEAD LOAD	SUPERIMPOSED DEAD 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.

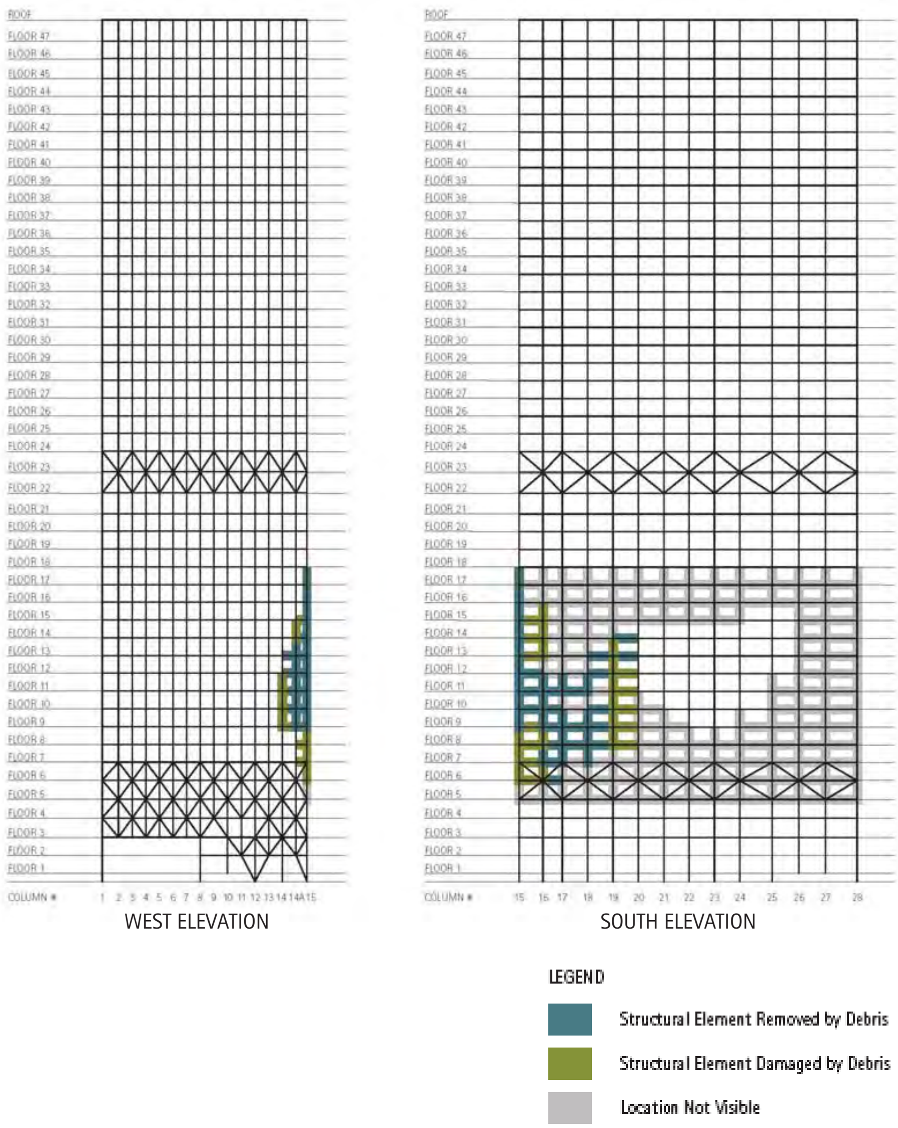


Figure 3.1 Debris damage to WTC7 from visual evidence documented in appendix of report by Dr Fred Mowrer (Ref 14)

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 (ϕ 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 STRENGTH	ULTIMATE 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

TYPE OF STEEL	SCALE FACTOR	YIELD STRENGTH	ULTIMATE 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.

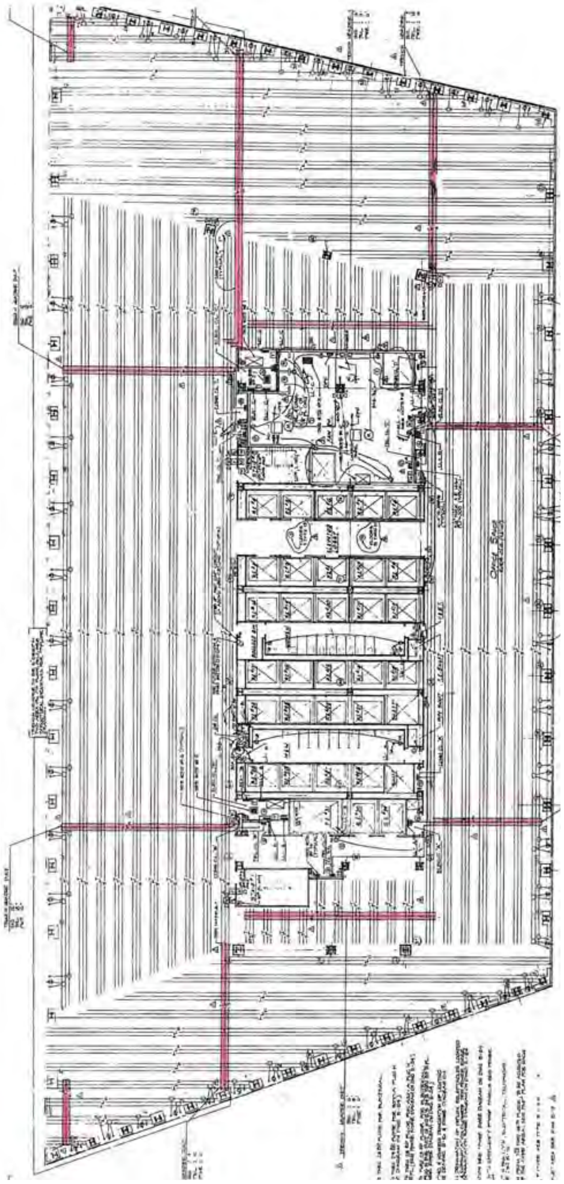


Figure 3.2 Location of trench header ducts (ie concrete slab discontinuities) on typical floors of WTC7 shown in red (PANYNJ0102202)

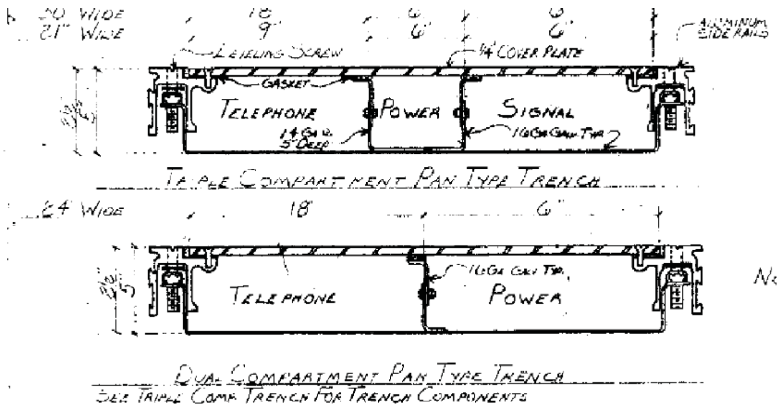


Figure 3.3 Trench header detail from MacFab WTC7 shop drawing T5 (CANTOR2005517)

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 ft^2, 1891 ft^2, 1363 ft^2 and 1410 ft^2 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.

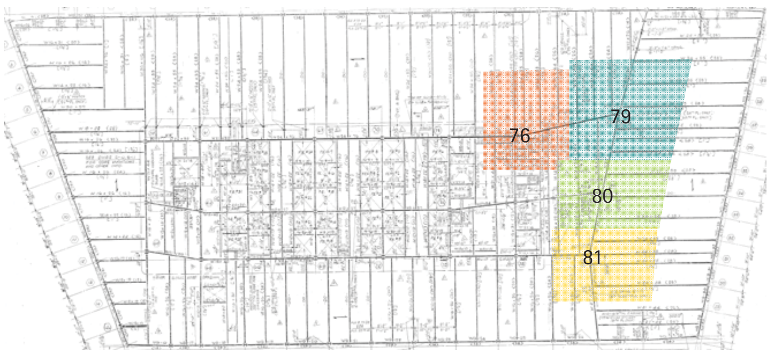


Figure 4.1 Approximate tributary zones of eastern interior columns

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.

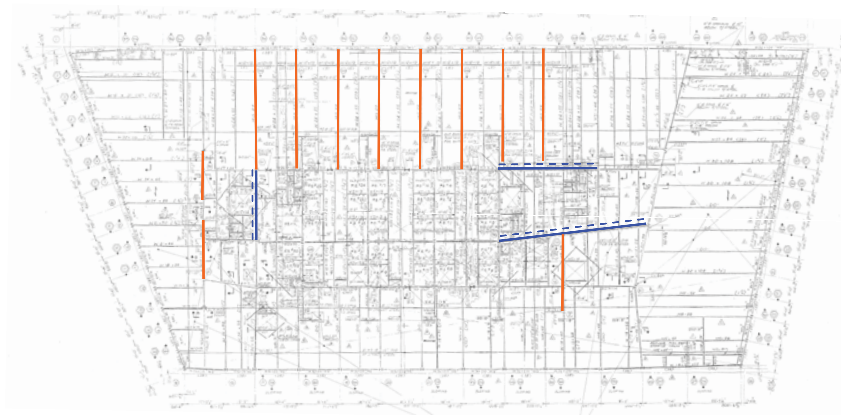


Figure 4.2 Transfer trusses (blue) and Transfer girders (orange) at Floor 7

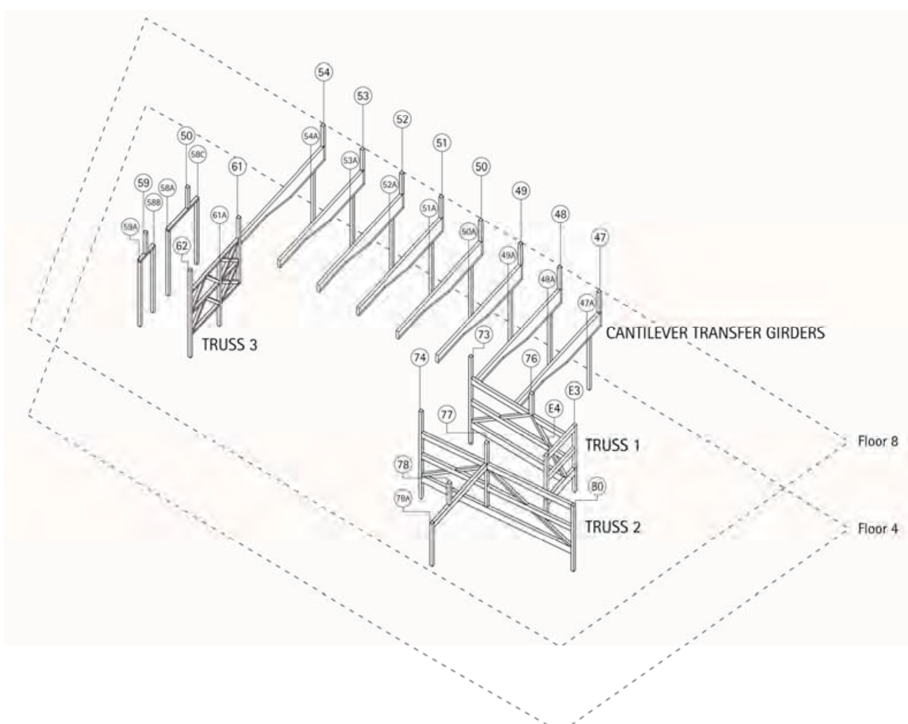


Figure 4.3 Transfer structures between Floors 5 and 7

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.

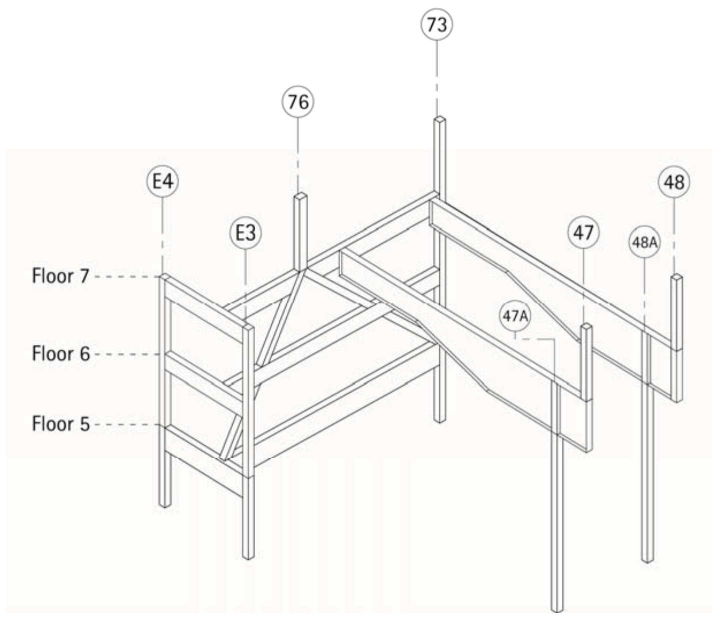


Figure 4.4 Example of interconnected transfer structures in WTC7 (Truss 1)

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.

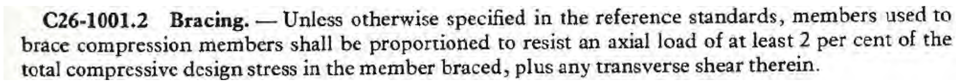
A floor diaphragm plays a critical role in connecting all structural components to a building's lateral system. Typically, the frames or walls of a building's lateral system are oriented and distributed in such a way that when they act together, they provide sufficiently stiff and strong lateral resistance 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.

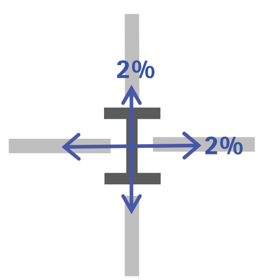


Figure 4.6 Diagram of column lateral bracing requirement in NYC Building Code

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).

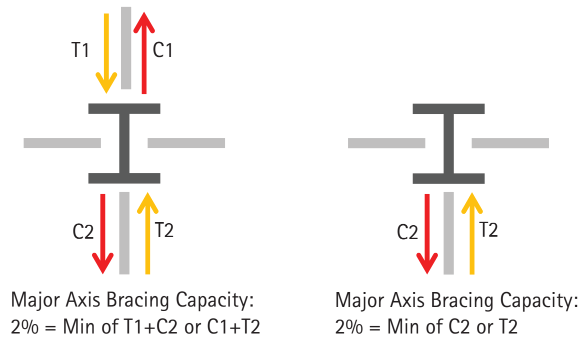


Figure 4.7a,b Four-sided and three-sided lateral bracing comparison (T=tension capacity of girder-to-column connection, C=compression capacity of girder-to-column connection)

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.

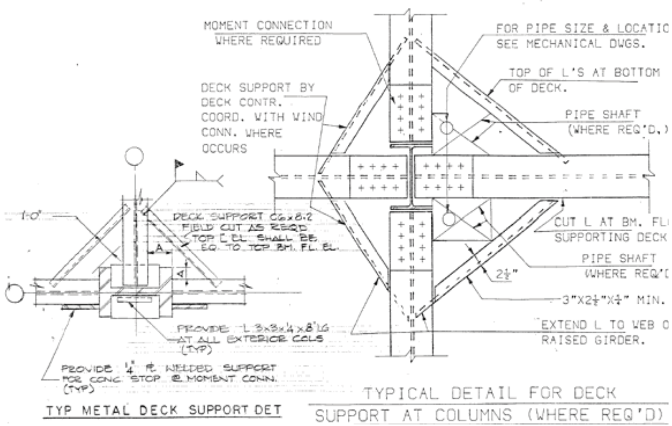


Figure 4.8 Typical detail on Sheet S24A of Cantor construction documents showing allowance for openings in slabs adjacent to interior columns (TISHMAN014742)

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).

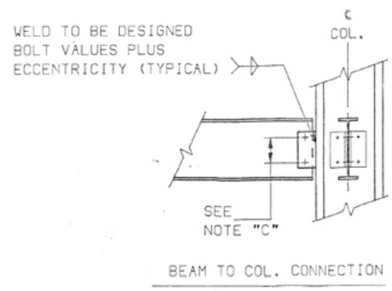


Figure 4.9 Typical beam-to-column connection detail on Sheet S-24 of the structural construction documents showing both shear tab and double angle connections (TISHMAN014741)

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.

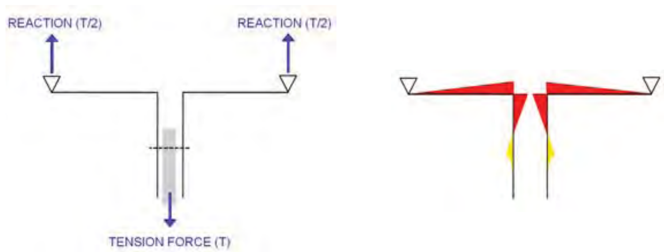


Figure 4.10 Behavior assumed for assessment of tension capacity of welded double-angle connections (free body diagram and bending moments)

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 LEVEL	2% TENSION FORCE DESIGN REQUIREMENT BASED ON DESIGN GRAVITY LOADS [KIPS]	CONNECTION TYPE	MAX TENSION CAPACITY USING L8x4x1 [KIPS]	MAX BRACING POSSIBLE (% OF REQUIRED)
Roof	6.8	Header Pl		
Floor 47	10.5	Header Pl		
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.

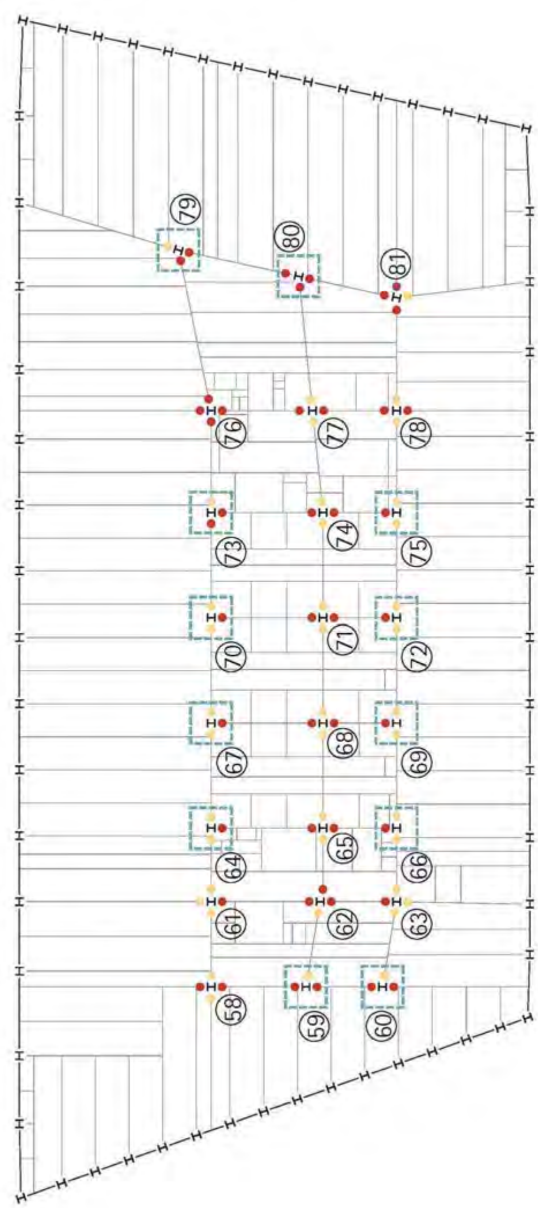


Figure 4.11 Typical WTC7 floor framing plan (red dots indicate knife connections which are vulnerable in tension; yellow dots indicate seated or header connections; blue boxes indicate interior columns with three-sided bracing)

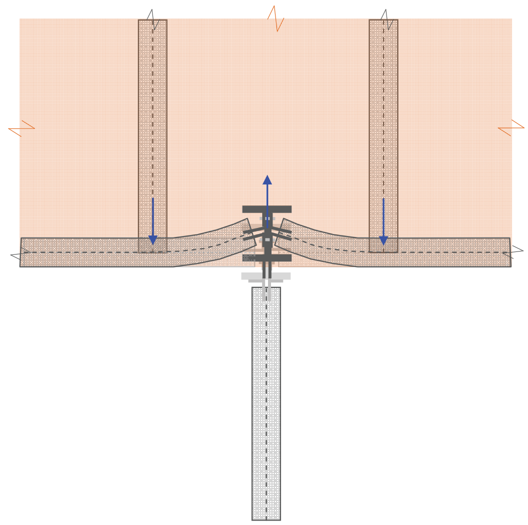


Figure 4.12 Diagram of accidental lateral bracing provided by minor axis bending of orthogonal girders when floor beams are present to provide reactions (configuration similar to Column 70 at core shown, deformations exaggerated)

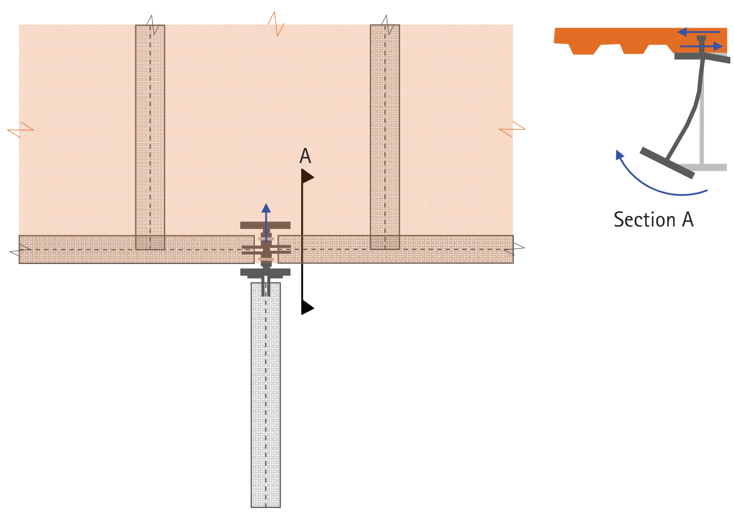


Figure 4.13 Diagram of inadvertent lateral bracing provided by torsional behavior of orthogonal girders when slab is intact (configuration similar to Column 70 at core shown, deformations exaggerated)

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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	Load in Column [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 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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	40.8	71.1	11.0
Floor 14	2100	42.0	71.1	11.0
Floor 13	2160	43.2	71.1	11.0
Floor 12	2220	44.4	71.1	11.0
Floor 11	2281	45.6	71.1	11.0
Floor 10	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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	2721	54.4	92.1	112.2
Floor 5	2883	57.7	116.2	69.5
Floor 4	2988	59.8	94.3	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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	72.6	50.1	158.7
Floor 8	3715	74.3	50.1	138.9
Floor 7	2349	47.0	NC	208.1
Floor 6	1001	20.0	NC	200.8
Floor 5	959	19.2	NC	223.1
Floor 4	1084	21.7	73.3	222.9
Floor 3	-	-	-	-
Floor 2	-	-	-	-

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

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column North-South Direction [Kip]	Total Bracing Capacity Provided to the Column East-West Direction [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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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 42	719	14.4	6.0	138.9
Floor 41	801	16.0	6.0	138.9
Floor 40	883	17.7	6.0	138.9
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 Bracing Code Check (no code violations)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	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

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

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	2076	41.5	224.9	NC
Floor 5	1718	34.4	NC	NC
Floor 4	1706	34.1	121.1	NC
Floor 3	2120	42.4	NC	NC
Floor 2	2390	47.8	200.4	NC

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

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column North-South Direction [Kip]	East-West Direction [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	-	-	-	-

Table 4.12 Column 68 Lateral Bracing Code Check (no code violations)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	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 Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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
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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [Kip]
Roof	105	2.1	69.0	138.9
Floor 47	170	3.4	94.3	178.4
Floor 46	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	140.4
Floor 5	1682	33.6	NC	298.3
Floor 4	1707	34.1	207.6	111.6
Floor 3	2232	44.6	NC	245.2
Floor 2	2262	45.2	321.5	147.5

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

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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
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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	2370	47.4	10.3	196.3
Floor 26	2492	49.8	10.3	196.3
Floor 25	2614	52.3	10.3	196.3
Floor 24	2740	54.8	12.4	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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	51.9	NC	NC
Floor 4	2470	49.4	121.1	NC
Floor 3	3363	67.3	NC	NC
Floor 2	3769	75.4	200.4	NC

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

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column: North-South Direction [Kip]	Total Bracing Capacity Provided to the Column: East-West Direction [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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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 14	2808	56.2	69.0	178.4
Floor 13	2897	57.9	69.0	178.4
Floor 12	2985	59.7	69.0	178.4
Floor 11	3073	61.5	69.0	178.4
Floor 10	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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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)

Floor Level	Design Compression Load in Column [Kip]	Required 2% Bracing Force [Kip]	Total Bracing Capacity Provided to the Column	
			North-South Direction [Kip]	East-West Direction [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	103.3	50.1	94.3
Floor 7	5463	109.3	270.7	10.0
Floor 6	5475	109.5	173.0	115.0
Floor 5	6041	120.8	262.4	166.9
Floor 4	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 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.	No event
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 upper-most 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 cont Summary of probable global collapse sequence

STAGE	INTERIOR EVENT	EXTERIOR EVENT
4 (Fig 5.6)	The eastern-most diagonals and supports of Transfer Trusses 1 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.	Following failure of the eastern diagonal of Transfer Truss 1, tension force 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 cantilevered transfer girders framing into it which support perimeter Columns 47 and 48. As a consequence, the 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 (Fig 5.7)	The falling floor areas tributary to Columns 76-78 impose an eastern horizontal force on the remaining intact floors to the west. The intact floors are susceptible to rupture due to their pre-segmentation by the trench headers and core openings, and the horizontal forces cause them to break apart in the horizontal plane at the boundaries of the trench headers. The resulting lateral displacements of the slab segments cause Columns 64-75 to lose stability above Floor 7. Simultaneously Column 81 buckles due to fracture of its east connections following the loss of the floor areas tributary to Column 78.	As the northeast corner of the perimeter frame buckles over its lower floors, the upper region of the frame sways northward creating a horizontal "kink" visible on the north façade of the building. The perimeter frame buckling at the base of the building spreads to the south and west as the weight of the perimeter walls shifts to adjacent stable supports, overloads them and causes them to fail in rapid succession.

Table 5.1 cont Summary of probable global collapse sequence

STAGE	INTERIOR EVENT	EXTERIOR EVENT
6 (Fig 5.8)	As Columns 64 through 75 and their tributary floor areas fall, the transfer girders at Floor 7 supporting the north façade fail.	The rapid western spread of perimeter buckling at the base of the building continues on both its 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.

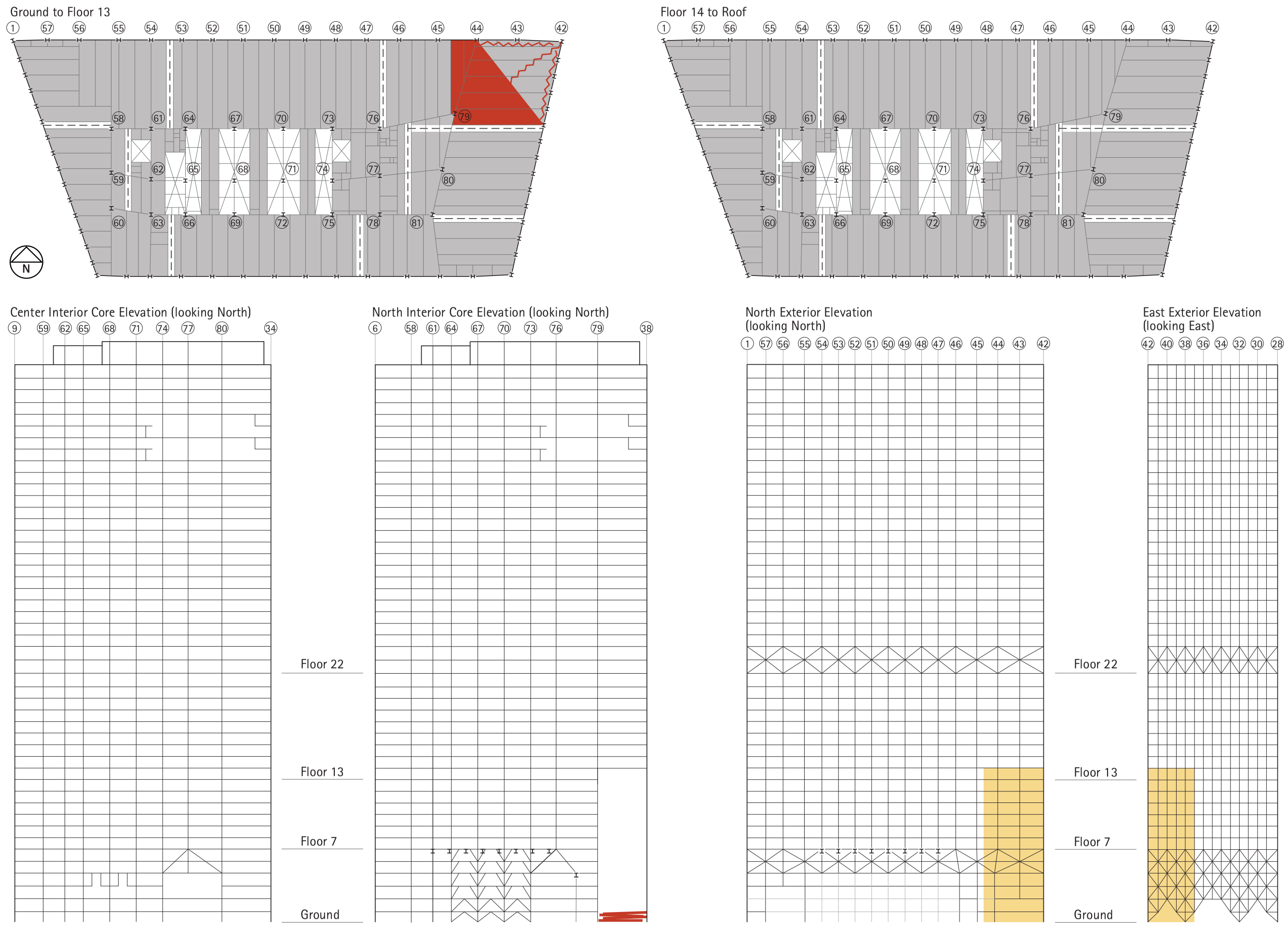


Figure 5.3 Stage 1 Scenario A Collapse Diagrams

PROBABLE GLOBAL COLLAPSE SEQUENCE STAGE 1 (SCENARIO A)

INTERIOR EVENTS - Following the unseating of Girder 44-79 at Floor 13, the floor framing and floor slabs supported by the girder 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.

EXTERIOR EVENTS - No Event

NOTE - Deformations not to scale

LEGEND

- Structural Failure
- Exterior Frame Unbraced
- Floor Structure Removed
- Region Void
- Trench Header

Approximate Timing: Prior to visual evidence

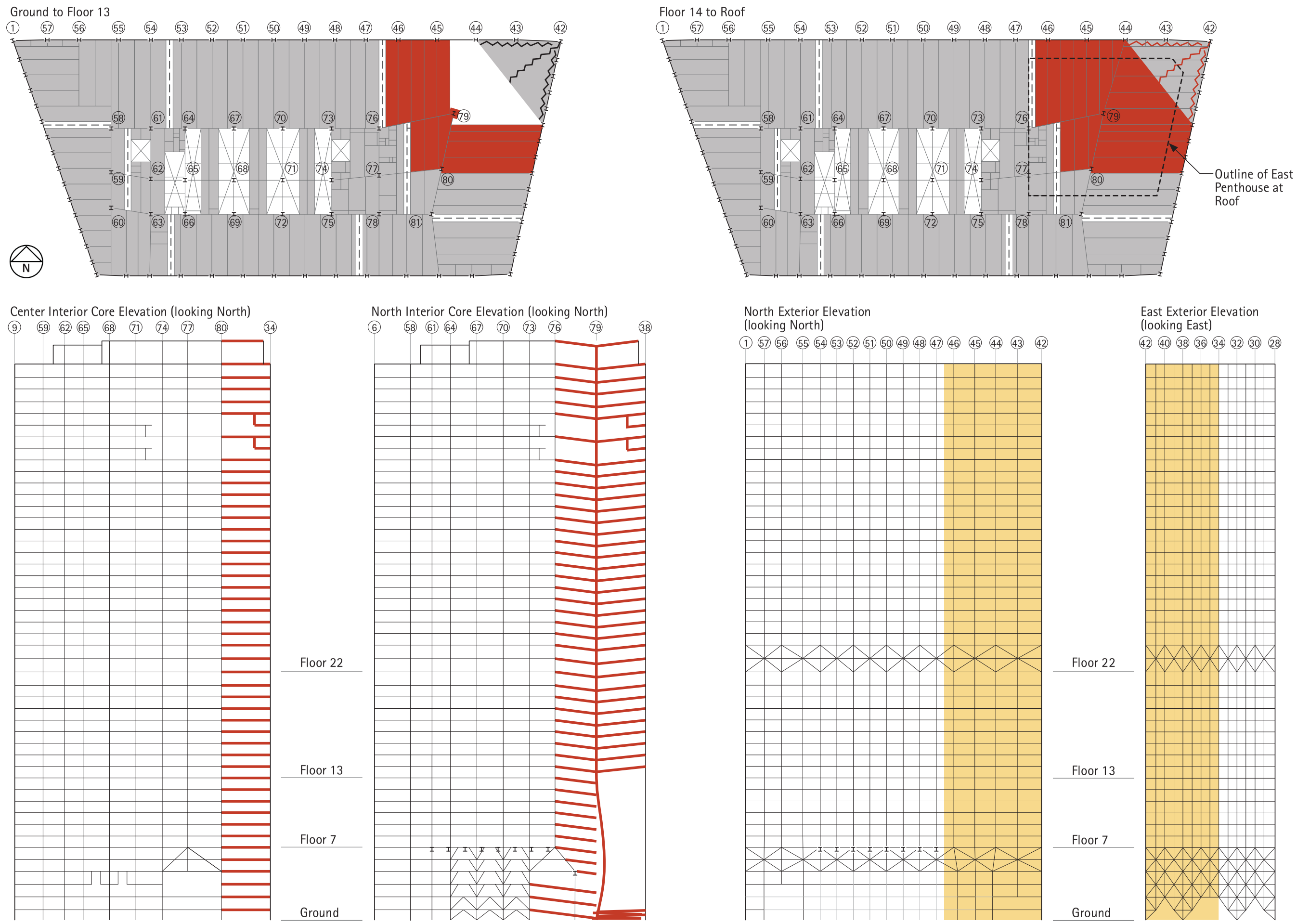


Figure 5.4 Stage 2 Scenario A Collapse Diagrams

PROBABLE GLOBAL COLLAPSE SEQUENCE STAGE 2 (SCENARIO A)

INTERIOR EVENTS - 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.

EXTERIOR EVENTS - No Event

NOTE - Deformations not to scale

LEGEND

- Structural Failure
- Exterior Frame Unbraced
- Floor Structure Removed
- Region Void
- Trench Header

Approximate Timing: During East Penthouse collapse

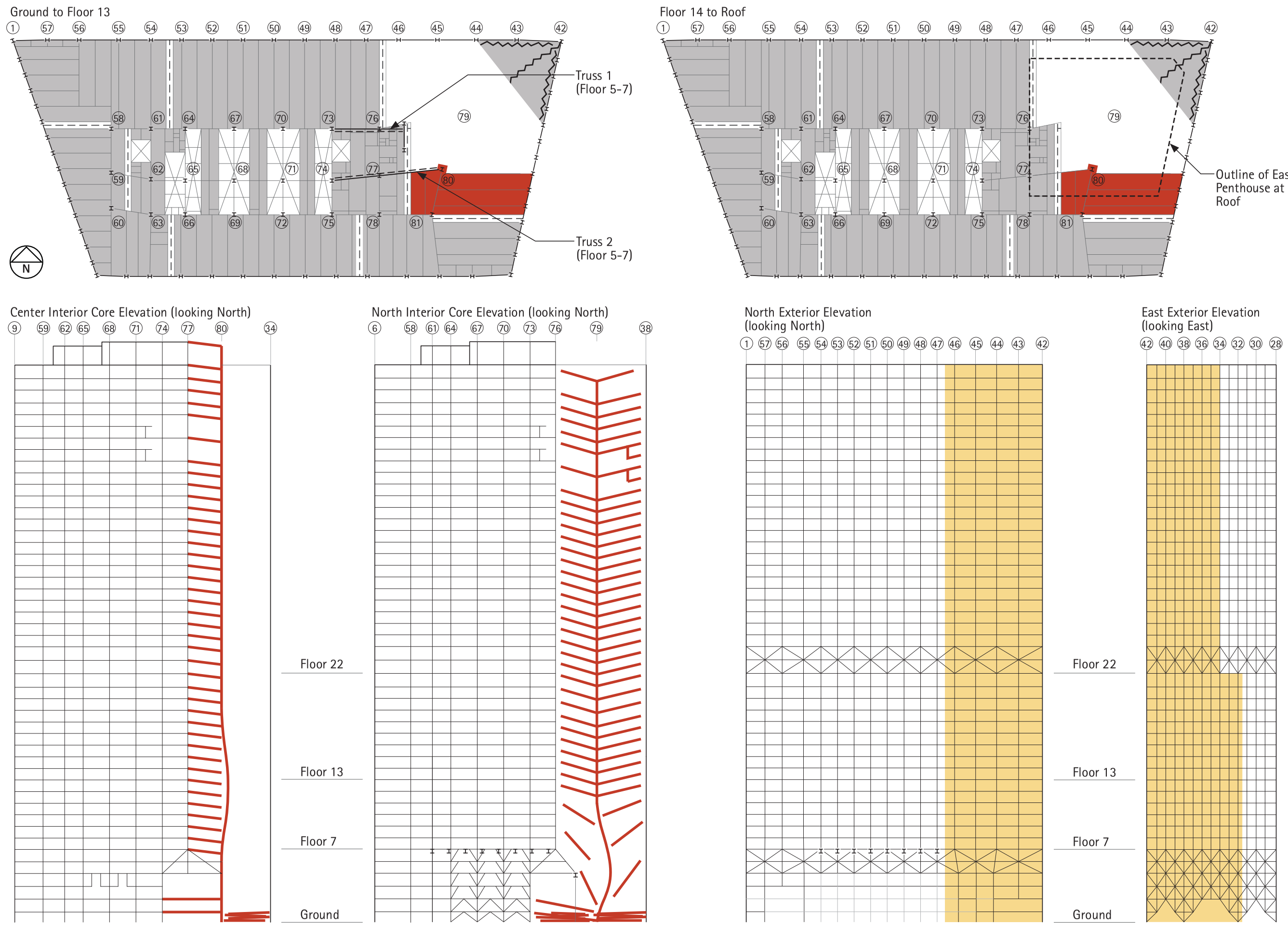


Figure 5.5 Stage 3 Collapse Diagrams

PROBABLE GLOBAL COLLAPSE SEQUENCE STAGE 3

INTERIOR EVENTS - 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 upper-most 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.

EXTERIOR EVENTS - No Event

NOTE - Deformations not to scale

LEGEND

- Structural Failure
- Exterior Frame Unbraced
- Floor Structure Removed
- Region Void
- Trench Header

Approximate Timing: During East Penthouse collapse and approximate 5-second pause

JA-3977

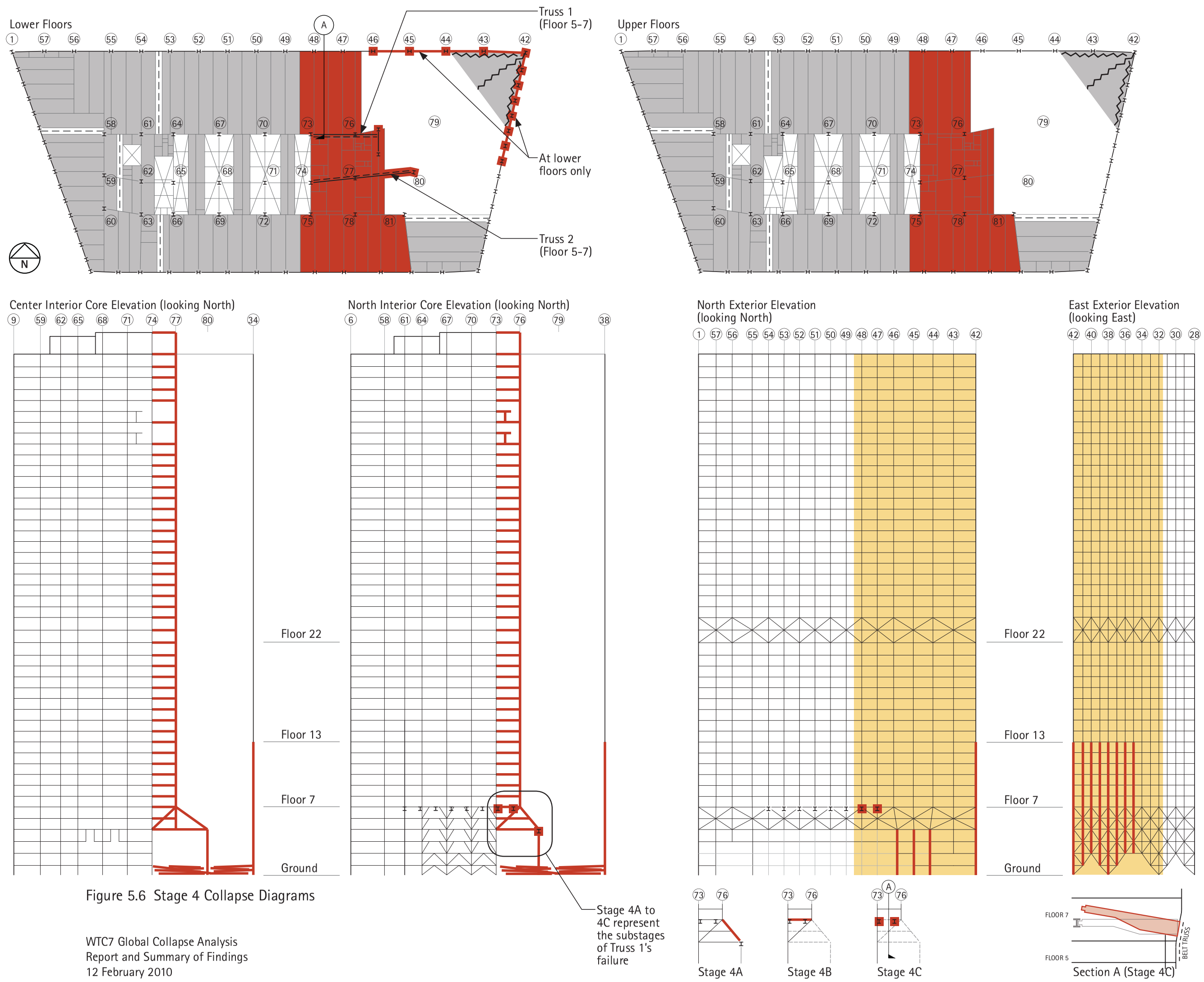


Figure 5.6 Stage 4 Collapse Diagrams

PROBABLE GLOBAL COLLAPSE SEQUENCE
STAGE 4

INTERIOR EVENTS - The eastern-most diagonals and supports of Transfer Trusses 1 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.

EXTERIOR EVENTS - Following failure of the eastern diagonal of Transfer Truss 1, tension force 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 cantilevered transfer girders framing into it which support perimeter Columns 47 and 48. As a consequence, the 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.

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

LEGEND

- Structural Failure
- Exterior Frame Unbraced
- Floor Structure Removed
- Region Void
- Trench Header

Approximate Timing: At onset of collapse of remaining interior structure

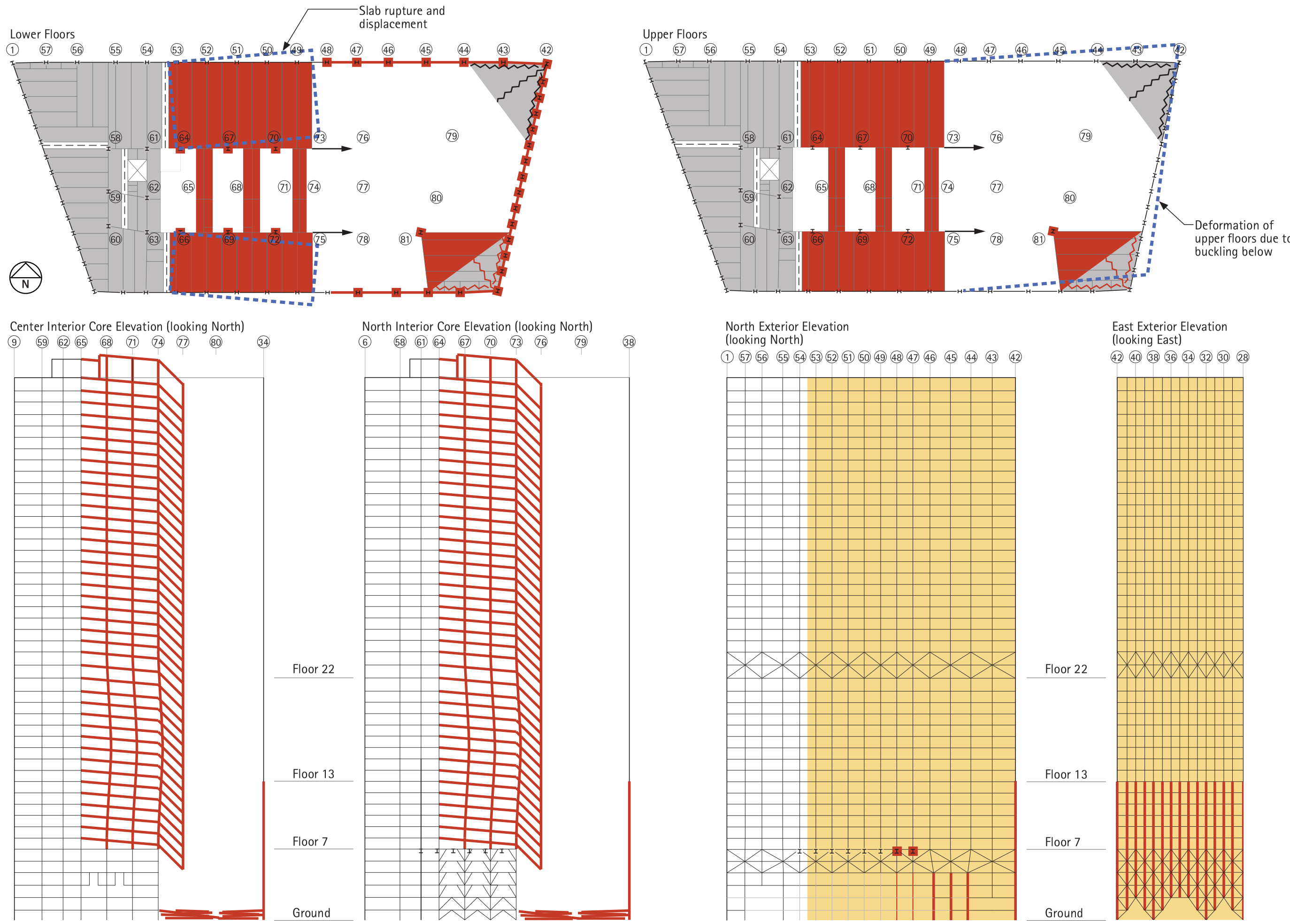


Figure 5.7 Stage 5 Collapse Diagrams

PROBABLE GLOBAL COLLAPSE SEQUENCE
STAGE 5

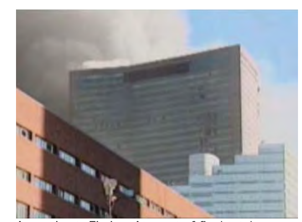
INTERIOR EVENTS - The falling floor areas tributary to Columns 76-78 impose an eastern horizontal force on the remaining intact floors to the west. The intact floors are susceptible to rupture due to their pre-segmentation by the trench headers and core openings, and the horizontal forces cause them to break apart in the horizontal plane at the boundaries of the trench headers. The resulting lateral displacements of the slab segments cause Columns 64-75 to lose stability above Floor 7. Simultaneously Column 81 buckles due to fracture of its east connections following the loss of the floor areas tributary to Column 78.

EXTERIOR EVENTS - As the northeast corner of the perimeter frame buckles over its lower floors, the upper region of the frame sways northward creating a horizontal "kink" visible on the north façade of the building. The perimeter frame buckling at the base of the building spreads to the south and west as the weight of the perimeter walls shifts to adjacent stable supports, overloads them and causes them to fail in rapid succession.

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

LEGEND

- Structural Failure
- Exterior Frame Unbraced
- Floor Structure Removed
- Region Void
- Trench Header



Approximate Timing: At onset of final total collapse (horizontal "kink")

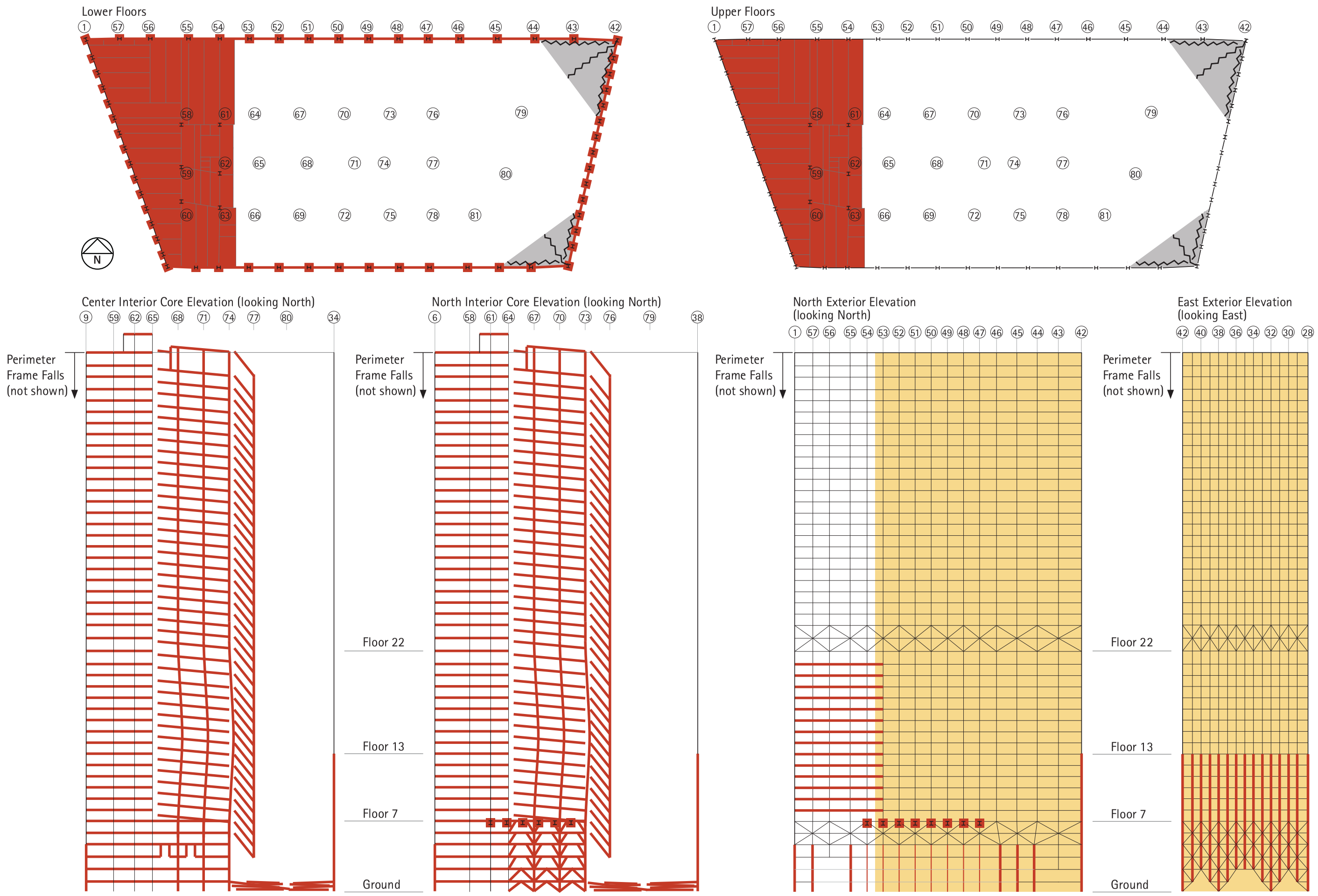


Figure 5.8 Stage 6 Collapse Diagrams

PROBABLE GLOBAL COLLAPSE SEQUENCE
STAGE 6

INTERIOR EVENTS - As Columns 64 through 75 and their tributary floor areas fall, the transfer girders at Floor 7 supporting the north façade fail.

EXTERIOR EVENTS - The rapid western spread of perimeter buckling at the base of the building continues on both its 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.

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

LEGEND

- Structural Failure
- Exterior Frame Unbraced
- Floor Structure Removed
- Region Void
- Trench Header

Approximate Timing: During final total collapse

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:

((Potential Energy of Falling Floor) – (Energy Dissipated in Failure of Floor)) VS (Energy Required to Fail Girder Connection to Column at Floor Below)

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).

STEP 1: ((Potential Energy of Floor 13 Falling to Floor 12) – (Energy Dissipated in Failure of Floor 13)) > (Energy Required to Fail Floor 12 Girder Connection to Column) → Floor Collapse Propagates

STEP 2: ((Potential Energy of Floor 12 Falling to Floor 11) – (Energy Dissipated in Failure of Floor 12)) > (Energy Required to Fail Floor 11 Girder Connection to Column) → Floor Collapse Propagates

STEP 3: ((Potential Energy of Floor 11 Falling to Floor 10) – (Energy Dissipated in Failure of Floor 11)) > (Energy Required to Fail Floor 10 Girder Connection to Column) → Floor Collapse Propagates

ETC, TO GROUND

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).

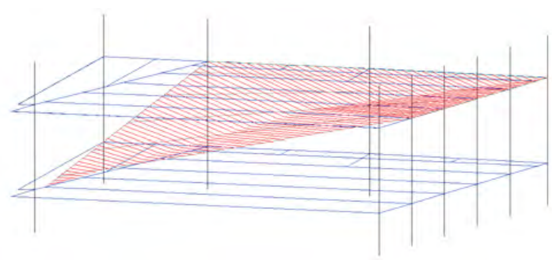


Figure 5.10 Deformed geometry of floor during collapse

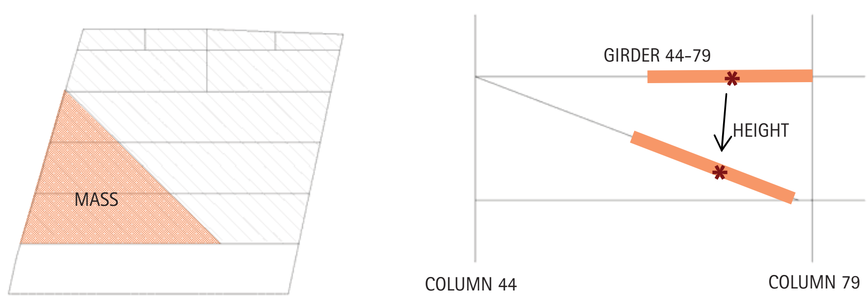


Figure 5.11 Basis for potential energy calculation at each floor level

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

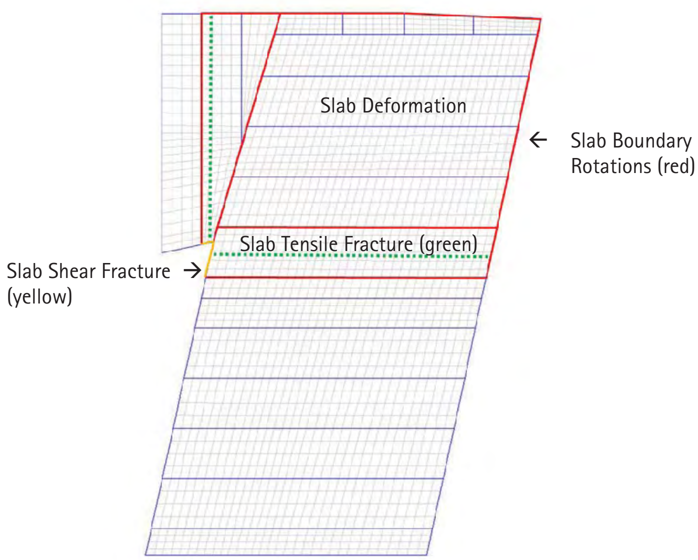


Figure 5.12 Assumed energy dissipation modes and locations (slab boundary rotations on southern boundary of floor failure only included where full-depth trench headers not present)

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 pre-segmentation 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 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).

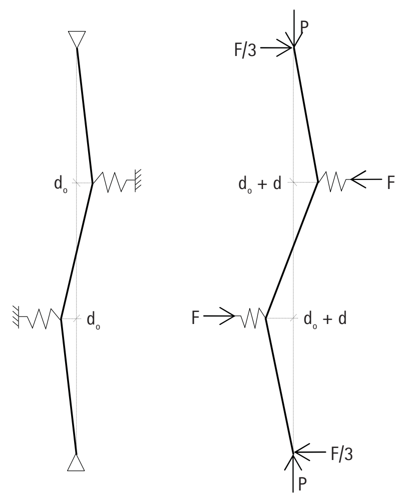


Figure 5.13 Example of lateral forces resulting from an initial crookedness of a column (Ref 15)

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.

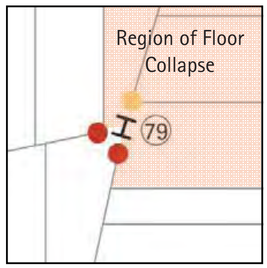


Figure 5.14 Remaining bracing configuration for Column 79 following floor failure (red = welded double-angle knife connections which are weak in tension)

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.

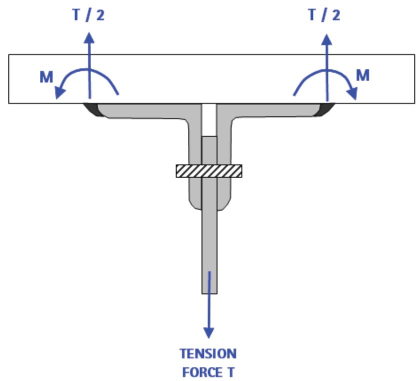


Figure 5.15 Combined tension and moment demands on fillet welds of knife connection due to tensile force

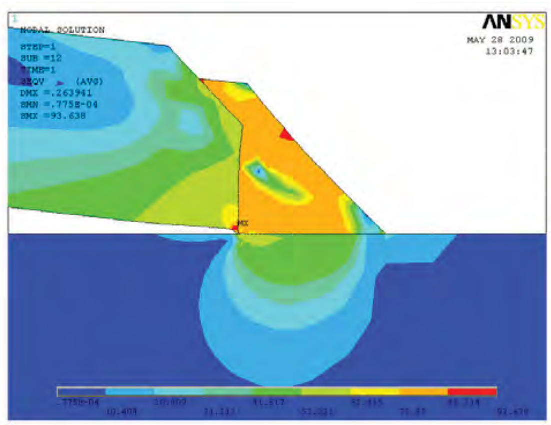


Figure 5.16 Stress concentration at root of fillet weld due to tension force on knife connection (image from report by Dr Anthony Ingraffea, Appendix A)

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.

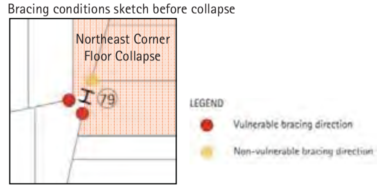
Figure 5.17 Input and Output for Scenario A Column 79 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

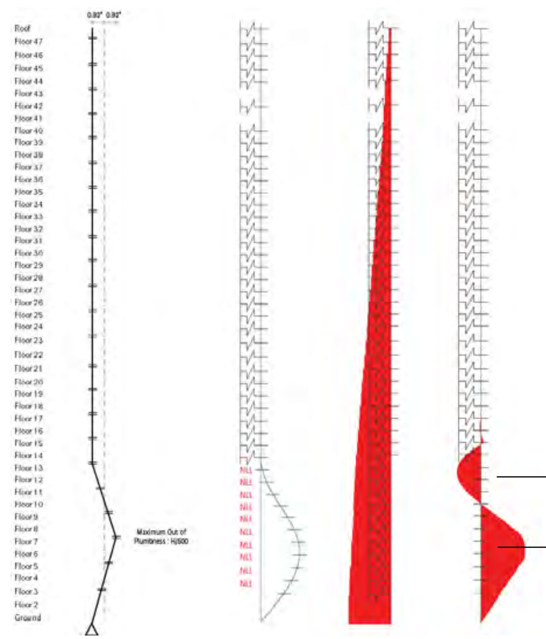
Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	SUSTAINED AXIAL COLUMN LOADS [kip]	FRAME SECTIONS	COLUMN BRACING PROPERTIES IN THE MODEL			
			North	South	East	West
Roof	-254.2	W14x99	LS	LS	-	LS
Floor 47	-417.4	W14x132	LS	LS	-	LS
Floor 46	-581.3		LS	LS	-	LS
Floor 45	-754.2	W14x193	LS	LS	-	LS
Floor 44	-943.2		LS	LS	-	LS
Floor 43	-945.9	W14x257	-	-	-	-
Floor 42	-1136.7		LS	LS	-	LS
Floor 41	-1140.1	W14x311	-	-	-	-
Floor 40	-1315.5		LS	LS	-	LS
Floor 39	-1490.1	W14x370	LS	LS	-	LS
Floor 38	-1665.4		LS	LS	-	LS
Floor 37	-1840.8	W14x426	LS	LS	-	LS
Floor 36	-2016.9		LS	LS	-	LS
Floor 35	-2193.0	W14x455	LS	LS	-	LS
Floor 34	-2369.3		LS	LS	-	LS
Floor 33	-2545.7	W14x500	LS	LS	-	LS
Floor 32	-2722.7		LS	LS	-	LS
Floor 31	-2899.9	W14x550	LS	LS	-	LS
Floor 30	-3077.7		LS	LS	-	LS
Floor 29	-3255.7	W14x605	LS	LS	-	LS
Floor 28	-3434.3		LS	LS	-	LS
Floor 27	-3613.1	W14x665	LS	LS	-	LS
Floor 26	-3792.7		LS	LS	-	LS
Floor 25	-3972.4	W14x730	LS	LS	-	LS
Floor 24	-4153.3		LS	LS	-	LS
Floor 23	-4377.0	W14x730 + 2 SIDE Pl's 26x1	LS	LS	-	LS
Floor 22	-4603.1		LS	LS	-	LS
Floor 21	-4829.3	W14x730 + 2 SIDE Pl's 26x1	LS	LS	-	LS
Floor 20	-5011.8		LS	LS	-	LS
Floor 19	-5194.4	W14x730 + 2 SIDE Pl's 26x1	LS	LS	-	LS
Floor 18	-5377.9		LS	LS	-	LS
Floor 17	-5561.9	W14x730 + 2 SIDE Pl's 26x2	LS	LS	-	LS
Floor 16	-5747.9		LS	LS	-	LS
Floor 15	-5934.0	W14x730 + 2 SIDE Pl's 26x2	LS	LS	-	LS
Floor 14	-6120.2		LS	LS	-	LS
Floor 13 *	-6229.5	W14x730 + 2 SIDE Pl's 26x2	-	LS	-	NLL (20.8kip)
Floor 12 *	-6338.8		-	LS	-	NLL (20.8kip)
Floor 11 *	-6448.4	W14x730 + 2 SIDE Pl's 26x3	-	LS	-	NLL (20.8kip)
Floor 10 *	-6559.5		-	LS	-	NLL (20.8kip)
Floor 9 *	-6671.1	W14x730 + 2 SIDE Pl's 26x3	-	LS	-	NLL (20.8kip)
Floor 8 *	-6782.7		-	LS	-	NLL (20.8kip)
Floor 7 *	-6923.3	W14x730 + 2 SIDE Pl's 26x3	-	LS	-	NLL (9.8kip)
Floor 6 *	-6974.4		-	LS	-	NLL (10.2kip)
Floor 5 *	-7168.2	W14x730 + 2 SIDE Pl's 26x4	-	LS	-	NLL (23.0kip)
Floor 4 *	-7301.6		-	LS	-	NLL (15.3kip)
Floor 3 *	-7382.7	W14x730 + 2 SIDE Pl's 26x5	-	LS	-	-
Floor 2 *	-7451.1		-	LS	-	-
Ground	-7451.1		-	-	-	-

* Floors at which a reduced load is assigned because of the northeast corner floor collapse

Bracing conditions sketch before collapse



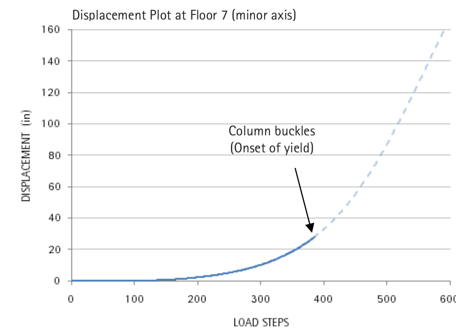


Initial geometry assumed per AISC Tolerances

Deformed Shape Under Sustained Loads

Axial Forces at Failure

Bending Moment (minor axis) at Failure



Cross Section: W14x730 + 2 Side Pl's 26x2
Axial Force demand: 6009 kip
Major Axis Moment Demand: 17 kip-in
Minor Axis Moment Demand: 35353 kip-in
Maximum Stress in Section: 45.5 ksi $< F_{ye} = 46$ ksi
(In side plate)

Cross Section: W14x730 + 2 Side Pl's 26x3
Axial Force Demand: 6731 kip
Major Axis Moment Demand: 24 kip-in
Minor Axis Moment Demand: 76546 kip-in
Governing Stress in Section: 60.0 ksi $> F_{ye} = 59.9$ksi
(In side plate)

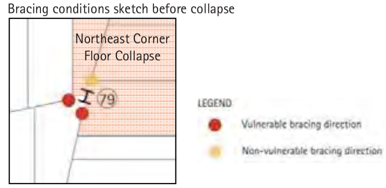
Figure 5.18 Input and Output for Scenario B Column 79 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

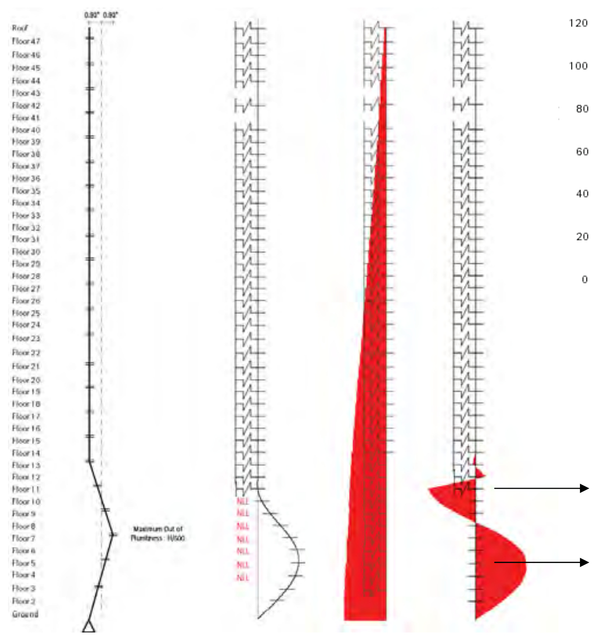
Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	SUSTAINED AXIAL COLUMN LOADS [kip]	FRAME SECTIONS	COLUMN BRACING PROPERTIES IN THE MODEL			
			North	South	East	West
Roof	-254.2	W14x99	LS	LS	-	LS
Floor 47	-417.4	W14x132	LS	LS	-	LS
Floor 46	-581.3		LS	LS	-	LS
Floor 45	-754.2	W14x193	LS	LS	-	LS
Floor 44	-943.2		LS	LS	-	LS
Floor 43	-945.9	W14x257	-	-	-	-
Floor 42	-1136.7		LS	LS	-	LS
Floor 41	-1140.1	W14x311	-	-	-	-
Floor 40	-1315.5		LS	LS	-	LS
Floor 39	-1490.1	W14x370	LS	LS	-	LS
Floor 38	-1665.4		LS	LS	-	LS
Floor 37	-1840.8	W14x426	LS	LS	-	LS
Floor 36	-2016.9		LS	LS	-	LS
Floor 35	-2193.0	W14x455	LS	LS	-	LS
Floor 34	-2369.3		LS	LS	-	LS
Floor 33	-2545.7	W14x500	LS	LS	-	LS
Floor 32	-2722.7		LS	LS	-	LS
Floor 31	-2899.9	W14x550	LS	LS	-	LS
Floor 30	-3077.7		LS	LS	-	LS
Floor 29	-3255.7	W14x605	LS	LS	-	LS
Floor 28	-3434.3		LS	LS	-	LS
Floor 27	-3613.1	W14x665	LS	LS	-	LS
Floor 26	-3792.7		LS	LS	-	LS
Floor 25	-3972.4	W14x730	LS	LS	-	LS
Floor 24	-4153.3		LS	LS	-	LS
Floor 23	-4377.0	W14x730 + 2 SIDE Pl's 26x1	LS	LS	-	LS
Floor 22	-4603.1		LS	LS	-	LS
Floor 21	-4829.3	W14x730 + 2 SIDE Pl's 26x1	LS	LS	-	LS
Floor 20	-5011.8		LS	LS	-	LS
Floor 19	-5194.4	W14x730 + 2 SIDE Pl's 26x1	LS	LS	-	LS
Floor 18	-5377.9		LS	LS	-	LS
Floor 17	-5561.9	W14x730 + 2 SIDE Pl's 26x2	LS	LS	-	LS
Floor 16	-5747.9		LS	LS	-	LS
Floor 15	-5934.0	W14x730 + 2 SIDE Pl's 26x2	LS	LS	-	LS
Floor 14	-6120.2		LS	LS	-	LS
Floor 13	-6306.2	W14x730 + 2 SIDE Pl's 26x2	LS	LS	-	LS
Floor 12	-6492.2		LS	LS	-	LS
Floor 11	-6678.2	W14x730 + 2 SIDE Pl's 26x3	LS	LS	-	LS
Floor 10 *	-6789.3		-	LS	-	NLL (20.8kip)
Floor 9 *	-6900.9	W14x730 + 2 SIDE Pl's 26x3	-	LS	-	NLL (20.8kip)
Floor 8 *	-7012.6		-	LS	-	NLL (20.8kip)
Floor 7 *	-7153.1	W14x730 + 2 SIDE Pl's 26x3	-	LS	-	NLL (9.8kip)
Floor 6 *	-7204.3		-	LS	-	NLL (10.2kip)
Floor 5 *	-7398.0	W14x730 + 2 SIDE Pl's 26x4	-	LS	-	NLL (23.0kip)
Floor 4 *	-7531.4		-	LS	-	NLL (15.3kip)
Floor 3 *	-7612.5	W14x730 + 2 SIDE Pl's 26x5	-	LS	-	-
Floor 2 *	-7681.0		-	LS	-	-
Ground	-7681.0		-	-	-	-

* Floors at which a reduced load is assigned because of the northeast corner floor collapse

Bracing conditions sketch before collapse



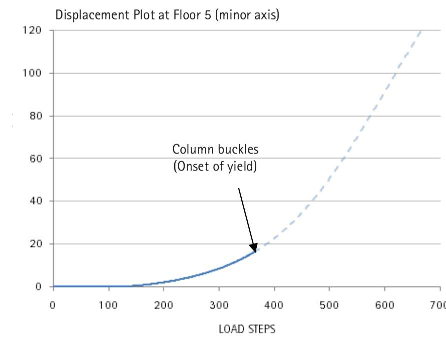


Initial geometry assumed per AISC Tolerances

Deformed Shape Under Sustained Loads

Axial Forces at Failure

Bending Moment (minor axis) at Failure



Cross Section: W14x730 + 2 Side Pl's 26x2
Axial Force demand: 6419 kip
Major Axis Moment Demand: 2 kip-in
Minor Axis Moment Demand: 34565 kip-in
Governing Stress in Section: 46.2 ksi > F_{ye} = 46.0 ksi
(In side plate)

Cross Section: W14x730 + 2 Side Pl's 26x3
Axial Force Demand: 7122 kip
Major Axis Moment Demand: 39 kip-in
Minor Axis Moment Demand: 62072 kip-in
Maximum Stress in Section: 53.2 ksi < F_{ye} = 59.9ksi
(In side plate)

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.

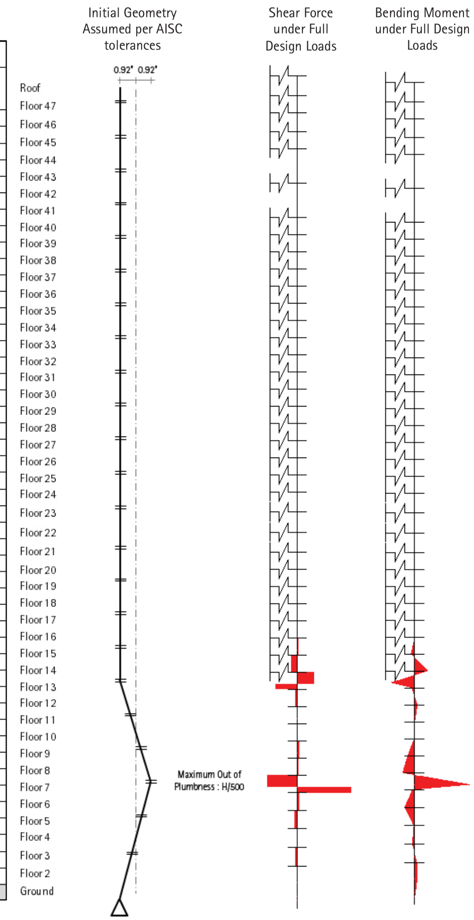
An additional stability analysis was conducted on Column 79 to demonstrate that had the column been provided with code-compliant lateral bracing, it would not have buckled following the loss of the lower floors in the northeast corner of the building. The analysis used the same methodology described 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).

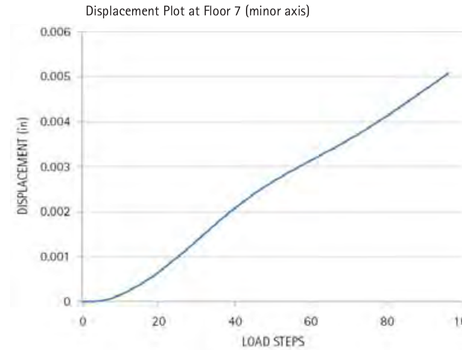
From the figure it is evident that after the application of the full design load on the column, no links have failed and as a result, the bending moments in the column are very low. The displacement plot shows nearly linear response up to the full axial design load of 7962 kips at Floor 7. The maximum 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).

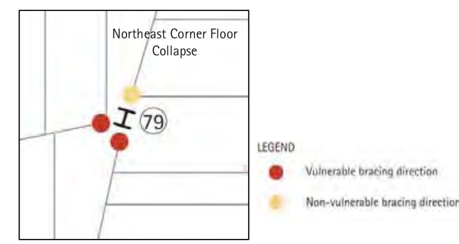
Figure 5.19 Input and Output for Stability Analysis for Column 79 (Scenario A) with 2% Bracing Capacity Illustrating Column Stability Under Sustained Gravity Loads

Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	FRAME SECTIONS	DESIGN AXIAL COLUMN LOADS [kip]	REQ CONN AXIAL CAPACITY (2%)	COLUMN BRACING PROPERTIES			
				North/South Direction	Link Force under Full Design Load	East/West Direction	Link Force under Full Design Load
Roof	W14x99	-341.7	6.83	LS	-	LS	-
Floor 47	W14x132	-523.8	10.48	LS	-	LS	-
Floor 46		-702.9	14.06	LS	-	LS	-
Floor 45	W14x193	-884.9	17.70	LS	-	LS	-
Floor 44		-1084.1	21.68	LS	-	LS	-
Floor 43	W14x257	-1086.8	21.74	-	-	-	-
Floor 42		-1287.6	25.75	LS	-	LS	-
Floor 41	W14x311	-1291.0	25.82	-	-	-	-
Floor 40		-1475.6	29.51	LS	-	LS	-
Floor 39	W14x370	-1659.3	33.19	LS	-	LS	-
Floor 38		-1843.7	36.87	LS	-	LS	-
Floor 37	W14x426	-2028.2	40.56	LS	-	LS	-
Floor 36		-2213.3	44.27	LS	-	LS	-
Floor 35	W14x455	-2398.5	47.97	LS	-	LS	-
Floor 34		-2583.9	51.68	LS	-	LS	-
Floor 33	W14x500	-2769.4	55.39	LS	-	LS	-
Floor 32		-2955.6	59.11	LS	-	LS	-
Floor 31	W14x550	-3141.8	62.84	LS	-	LS	-
Floor 30		-3328.7	66.57	LS	-	LS	-
Floor 29	W14x605	-3515.8	70.32	LS	-	LS	-
Floor 28		-3703.6	74.07	LS	-	LS	-
Floor 27	W14x665	-3891.5	77.83	LS	-	LS	-
Floor 26		-4080.1	81.60	LS	-	LS	-
Floor 25	W14x730	-4269.0	85.38	LS	-	LS	-
Floor 24		-4459.0	89.18	LS	-	LS	-
Floor 23	W14x730 + 2 SIDE Pl 26x1	-4696.1	93.92	LS	-	LS	-
Floor 22		-4935.5	98.71	LS	-	LS	-
Floor 21	W14x730 + 2 SIDE Pl 26x1	-5174.8	103.50	LS	-	LS	-
Floor 20		-5366.2	107.32	LS	-	LS	-
Floor 19	W14x730 + 2 SIDE Pl 26x1	-5557.6	111.15	LS	-	LS	-
Floor 18		-5750.0	115.00	LS	-	LS	-
Floor 17	W14x730 + 2 SIDE Pl 26x2	-5942.9	118.86	LS	-	LS	-
Floor 16		-6137.8	122.76	LS	-	LS	-
Floor 15	W14x730 + 2 SIDE Pl 26x2	-6332.8	126.66	LS	-	LS	-
Floor 14		-6527.8	130.56	LS	-	LS	-
Floor 13	W14x730 + 2 SIDE Pl 26x2	-6722.8	134.46	LS	-	NLL	compression
Floor 12		-6918.0	138.36	LS	-	NLL	0.78
Floor 11	W14x730 + 2 SIDE Pl 26x3	-7113.6	142.27	LS	-	NLL	compression
Floor 10		-7310.4	146.21	LS	-	NLL	compression
Floor 9	W14x730 + 2 SIDE Pl 26x3	-7508.0	150.16	LS	-	NLL	compression
Floor 8		-7705.8	154.12	LS	-	NLL	11.03
Floor 7	W14x730 + 2 SIDE Pl's 26x3	-7962.2	159.24	LS	-	NLL	18.02
Floor 6		-8103.1	162.06	LS	-	NLL	1.92
Floor 5		-8500.4	170.01	LS	-	NLL	compression
Floor 4	W14x730 + 2 SIDE Pl's 26x4	-8796.7	175.93	LS	-	NLL	1.34
Floor 3		-9027.5	180.55	LS	-	-	-
Floor 2	W14x730 + 2 SIDE Pl's 26x5	-9220.5	184.41	-	-	-	-
Ground		-	-	-	-	-	-







Cross Section: W14x730 + 2 Side Pl 26x3
Max Moment (btw Floor 7 & 8): 809.2 kip.in << Bending Capacity
Max Shear Force (at Floor 7): 16.8 kip << 2% Capacity

COLUMN DOES NOT BUCKLE

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.

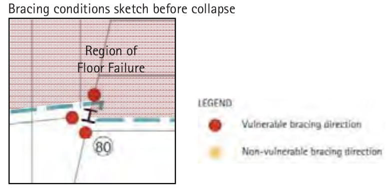
The results of the Column 80 stability analysis shown in Figure 5.20 are similar to those for Column 79. The western links representing the weak double-angle knife connections framing into the web plate of Column 80 begin to fail in tension under application of the sustained gravity loads. After a number 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.

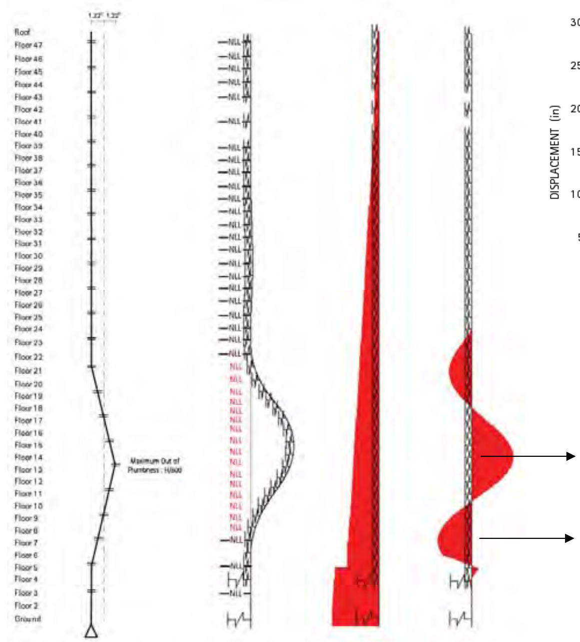
Figure 5.20 Input and Output for Column 80 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	SUSTAINED AXIAL COLUMN LOADS [kip]	FRAME SECTIONS	COLUMN BRACING PROPERTIES IN THE MODEL			
			North	South	East	West
Roof	-246.5	W14x90	-	NLL (12.3kip)	-	LS
Floor 47	-367.7	W14x99	-	NLL (12.3kip)	-	LS
Floor 46	-489.0		-	NLL (12.3kip)	-	LS
Floor 45	-610.3	W14x145	-	NLL (12.3kip)	-	LS
Floor 44	-739.2		-	NLL (12.3kip)	-	LS
Floor 43	-741.4	W14x193	-	-	-	-
Floor 42	-871.7		-	NLL (12.3kip)	-	LS
Floor 41	-874.4	W14x233	-	-	-	-
Floor 40	-997.5		-	NLL (12.3kip)	-	LS
Floor 39	-1119.8	W14x283	-	NLL (12.3kip)	-	LS
Floor 38	-1242.7		-	NLL (12.3kip)	-	LS
Floor 37	-1365.7	W14x322	-	NLL (12.3kip)	-	LS
Floor 36	-1489.3		-	NLL (12.3kip)	-	LS
Floor 35	-1613.1	W14x398	-	NLL (12.3kip)	-	LS
Floor 34	-1737.4		-	NLL (12.3kip)	-	LS
Floor 33	-1861.8	W14x455	-	NLL (12.3kip)	-	LS
Floor 32	-1986.9		-	NLL (12.3kip)	-	LS
Floor 31	-2112.0	W14x500	-	NLL (12.3kip)	-	LS
Floor 30	-2237.6		-	NLL (12.3kip)	-	LS
Floor 29	-2363.3	W14x550	-	NLL (12.3kip)	-	LS
Floor 28	-2489.5		-	NLL (12.3kip)	-	LS
Floor 27	-2615.7	W14x605	-	NLL (12.3kip)	-	LS
Floor 26	-2742.6		-	NLL (12.3kip)	-	LS
Floor 25	-2869.3	W14x665	-	NLL (12.3kip)	-	LS
Floor 24	-2997.1		-	NLL (12.3kip)	-	LS
Floor 23	-3153.5	W14x730	-	NLL (12.3kip)	-	LS
Floor 22	-3310.7		-	NLL (12.3kip)	-	LS
Floor 21	-3467.9	W14x730 + 2 WEB Pl 12.5x1	-	NLL (12.3kip)	-	LS
Floor 20	-3594.6		-	NLL (12.3kip)	-	LS
Floor 19	-3720.9	W14x730 + 2 WEB Pl 12.5x2	-	NLL (12.3kip)	-	LS
Floor 18	-3850.6		-	NLL (12.3kip)	-	LS
Floor 17	-3980.3	W14x730 + 2 WEB Pl 12.5x3	-	NLL (12.3kip)	-	LS
Floor 16	-4110.9		-	NLL (12.3kip)	-	LS
Floor 15	-4241.6	W14x730 + 2 WEB Pl 12.5x4	-	NLL (12.3kip)	-	LS
Floor 14	-4373.1		-	NLL (12.3kip)	-	LS
Floor 13	-4504.8	W14x730 + 2 WEB Pl 12.5x5	-	NLL (12.3kip)	-	LS
Floor 12	-4637.3		-	NLL (12.3kip)	-	LS
Floor 11	-4769.8	W14x730 + 2 WEB Pl 12.5x6	-	NLL (12.3kip)	-	LS
Floor 10	-4901.3		-	NLL (12.3kip)	-	LS
Floor 9	-5034.9	W14x730 + 2 WEB Pl 12.5x8	-	NLL (12.3kip)	-	LS
Floor 8	-5170.4		-	NLL (12.3kip)	-	LS
Floor 7	-5464.4	W14x730 + 2 WEB Pl 12.5x8	-	NLL (20.0kip)	-	LS
Floor 6	-5487.5		-	-	-	LS
Floor 5	-7553.1	W14x730 + 2 WEB Pl 12.5x8	-	NLL (20.4kip)	-	LS
Floor 4	-7748.1		-	LS	-	LS
Floor 3	-7938.7	W14x730 + 2 SIDE Pl 26x5	-	NLL (23.0kip)	-	-
Floor 2	-8061.3		-	-	-	-
Ground	-8061.3		-	LS	-	-

Bracing conditions sketch before collapse



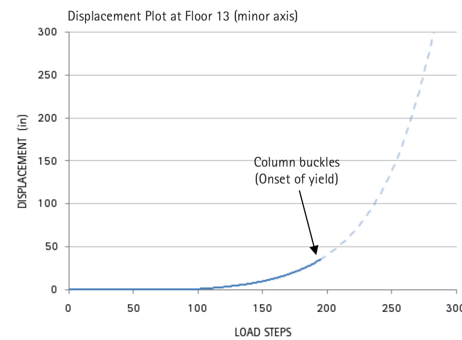


Initial geometry assumed per AISC Tolerances

Deformed Shape Under Sustained Loads

Axial Forces at Failure

Bending Moment (major axis) at Failure



Cross Section: W14x730 + 2 Web Pl 12.5x4
Axial Force demand: 4341 kip
Major Axis Moment Demand: 7197 kip-in
Minor Axis Moment Demand: 49674 kip-in
Governing Stress in Section: 60.0 ksi $> F_{ye} = 59.9$ ksi (In W-Section)

Cross Section: W14x730 + 2 Web Pl 12.5x8
Axial Force demand: 5427 kip
Major Axis Moment Demand: 8046kip-in
Minor Axis Moment Demand: 34145 kip-in
Maximum Stress in Section: 41.8 ksi $< F_{ye} = 59.9$ ksi (In web plate)

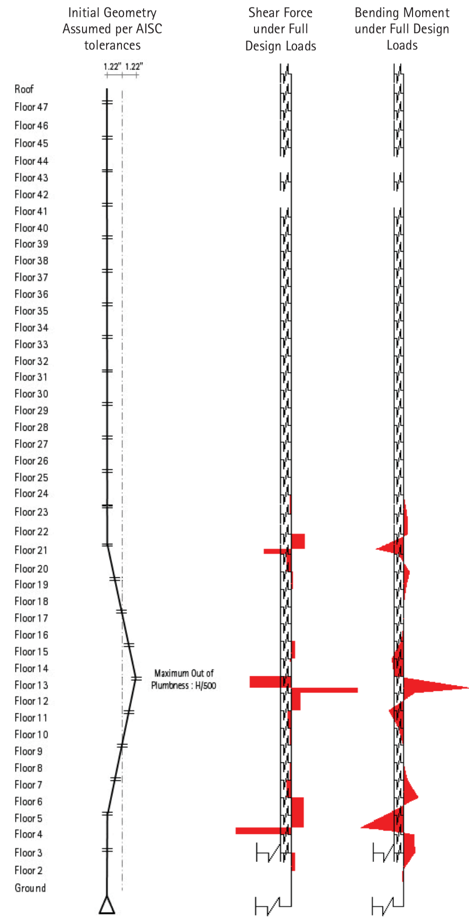
5.6.2 Stage 3 Collapse Prevention

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.

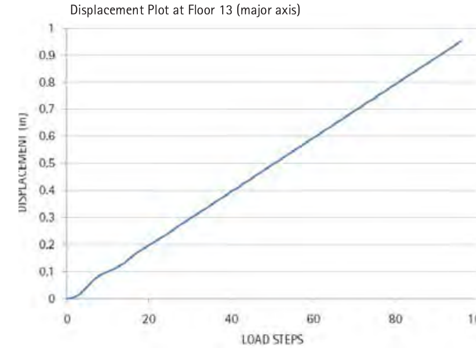
Figure 5.21 Input and Output for Stability Analysis for Column 80 with 2% Bracing Capacity Illustrating Column Stability Under Sustained Gravity Loads

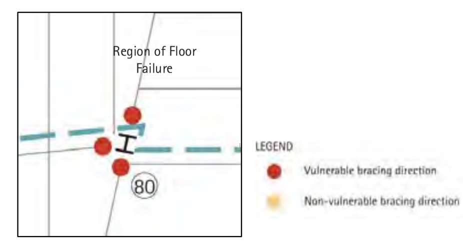
Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	FRAME SECTIONS	DESIGN AXIAL COLUMN LOADS [kip]	REQ CONN AXIAL CAPACITY (2%)	COLUMN BRACING PROPERTIES			
				North/South Direction	Link Force under Full Design Load	East/West Direction	Link Force under Full Design Load
Roof	W14x90	-313.9	6.28	NLL	Compression	LS	-
Floor 47	W14x99	-441.2	8.82	NLL	Compression	LS	-
Floor 46		-568.6	11.37	NLL	<0.01	LS	-
Floor 45	W14x145	-695.8	13.92	NLL	Compression	LS	-
Floor 44		-831.2	16.62	NLL	<0.01	LS	-
Floor 43	W14x193	-833.4	16.67	-	-	-	-
Floor 42		-970.1	19.40	NLL	<0.01	LS	-
Floor 41	W14x233	-972.9	19.46	-	-	-	-
Floor 40		-1101.9	22.04	NLL	<0.01	LS	-
Floor 39	W14x283	-1230.3	24.61	NLL	<0.01	LS	-
Floor 38		-1359.2	27.18	NLL	<0.01	LS	-
Floor 37	W14x322	-1488.2	29.76	NLL	<0.01	LS	-
Floor 36		-1617.8	32.36	NLL	<0.01	LS	-
Floor 35	W14x398	-1747.6	34.95	NLL	Compression	LS	-
Floor 34		-1878.0	37.56	NLL	<0.01	LS	-
Floor 33	W14x455	-2008.4	40.17	NLL	<0.01	LS	-
Floor 32		-2139.5	42.79	NLL	<0.01	LS	-
Floor 31	W14x500	-2270.6	45.41	NLL	<0.01	LS	-
Floor 30		-2402.2	48.04	NLL	<0.01	LS	-
Floor 29	W14x550	-2533.9	50.68	NLL	<0.01	LS	-
Floor 28		-2666.1	53.32	NLL	<0.01	LS	-
Floor 27	W14x605	-2798.4	55.97	NLL	<0.01	LS	-
Floor 26		-2931.2	58.62	NLL	<0.01	LS	-
Floor 25	W14x665	-3063.9	61.28	NLL	Compression	LS	-
Floor 24		-3197.8	63.96	NLL	0.05	LS	-
Floor 23	W14x730	-3363.1	67.26	NLL	0.23	LS	-
Floor 22		-3529.1	70.58	NLL	Compression	LS	-
Floor 21	W14x730 + 2 WEB PI 12.5x1	-3695.3	73.91	NLL	Compression	LS	-
Floor 20		-3827.8	76.56	NLL	0.27	LS	-
Floor 19	W14x730 + 2 WEB PI 12.5x2	-3960.0	79.20	NLL	Compression	LS	-
Floor 18		-4095.7	81.91	NLL	Compression	LS	-
Floor 17	W14x730 + 2 WEB PI 12.5x3	-4231.4	84.63	NLL	Compression	LS	-
Floor 16		-4368.0	87.36	NLL	Compression	LS	-
Floor 15	W14x730 + 2 WEB PI 12.5x4	-4504.7	90.09	NLL	0.81	LS	-
Floor 14		-4642.3	92.85	NLL	6.9	LS	-
Floor 13	W14x730 + 2 WEB PI 12.5x5	-4779.9	95.60	NLL	10.17	LS	-
Floor 12		-4918.4	98.37	NLL	2.85	LS	-
Floor 11	W14x730 + 2 WEB PI 12.5x6	-5056.9	101.14	NLL	Compression	LS	-
Floor 10		-5194.2	103.88	NLL	Compression	LS	-
Floor 9	W14x730 + 2 WEB PI 12.5x8	-5333.8	106.68	NLL	0.4	LS	-
Floor 8		-5475.2	109.50	NLL	0.81	LS	-
Floor 7	W14x730 + 2 WEB PI 12.5x8	-5775.0	115.50	NLL	Compression	LS	-
Floor 6		-5798.3	115.97	-	-	LS	-
Floor 5	W14x730 + 2 WEB PI 12.5x8	-7964.5	159.29	NLL	Compression	LS	-
Floor 4		-8168.2	163.36	LS	-	LS	-
Floor 3	W14x730 + 2 SIDE PI 26x5	-8367.5	167.35	NLL	Compression	-	-
Floor 2		-8498.1	169.96	-	-	-	-
Ground	-	-	-	LS	-	-	-



Displacement Plot at Floor 13 (major axis)





Cross Section: W14x730 + 2Web PI 12.5x5
Max Moment (btw Floor 13 & 14): 654 kip.in << Bending Capacity
Max Shear Force (at Floor 13): 11.5 kip << 2% Capacity

COLUMN DOES NOT BUCKLE

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.

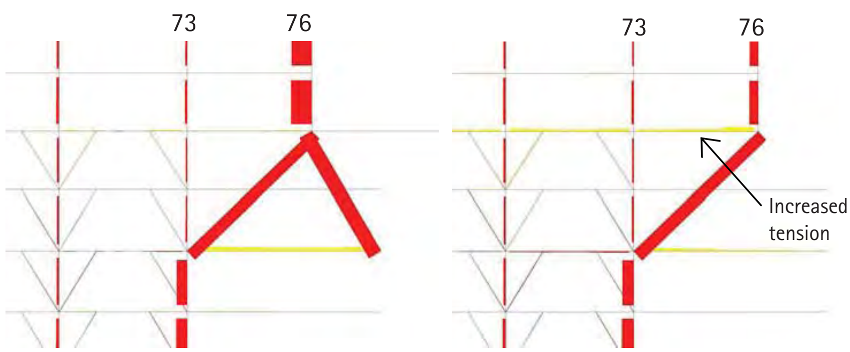


Figure 5.22 Transfer Truss 1 (view towards south) before and after east diagonal failure

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.

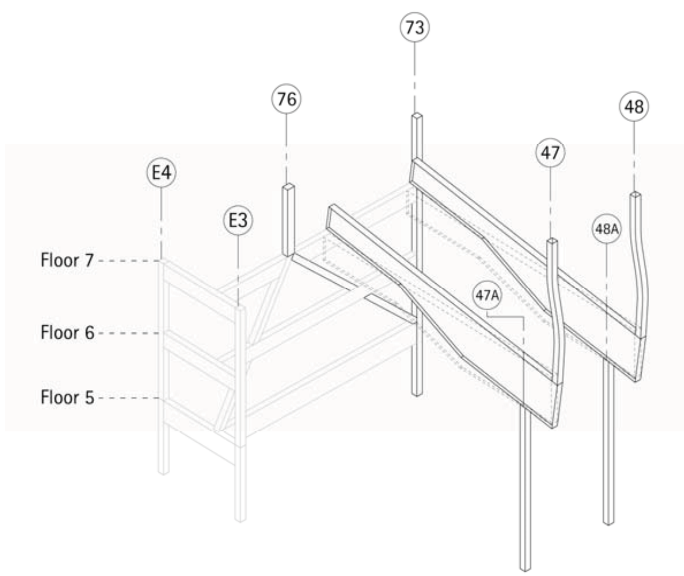


Figure 5.23 Rotation of Transfer Girders at Columns 47 and 48 following failure of top chord of Transfer Truss 1

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.

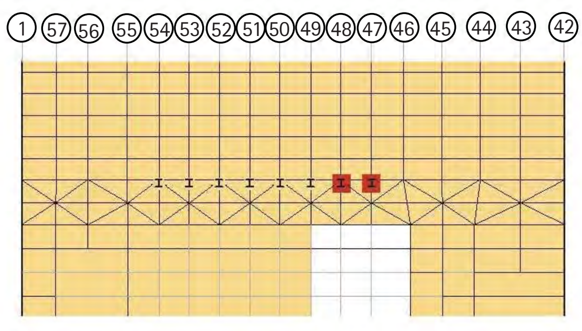


Figure 5.24 Bridging of north perimeter frame at Stage 4 between Columns 46 and 49 (failed transfer girders shown in red)

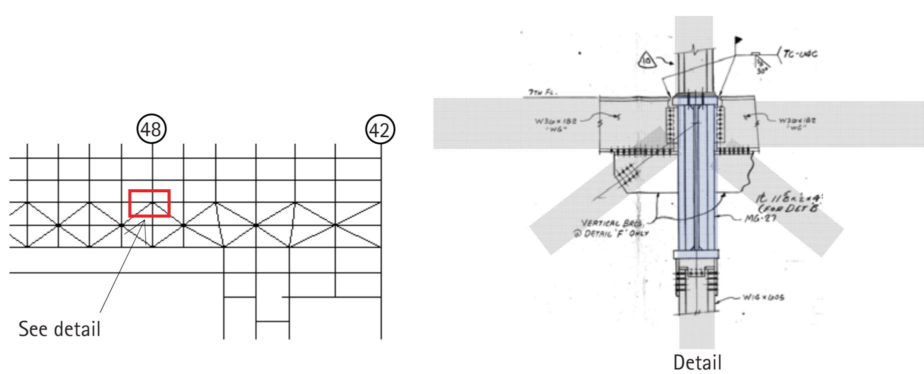
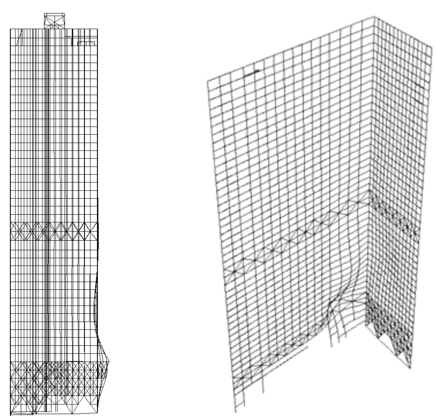


Figure 5.25 Detail from Frankel Steel erection drawing E119 showing discontinuity of perimeter belt truss members at connection to deep cantilevered girders (north elevation)

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.



(a) (b)

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

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.

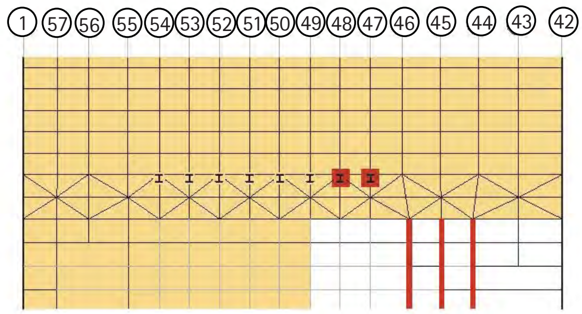


Figure 5.27 Bridging of north perimeter frame at Stage 4 between Columns 42 and 49 (failed transfer girders and buckled perimeter columns shown in red)

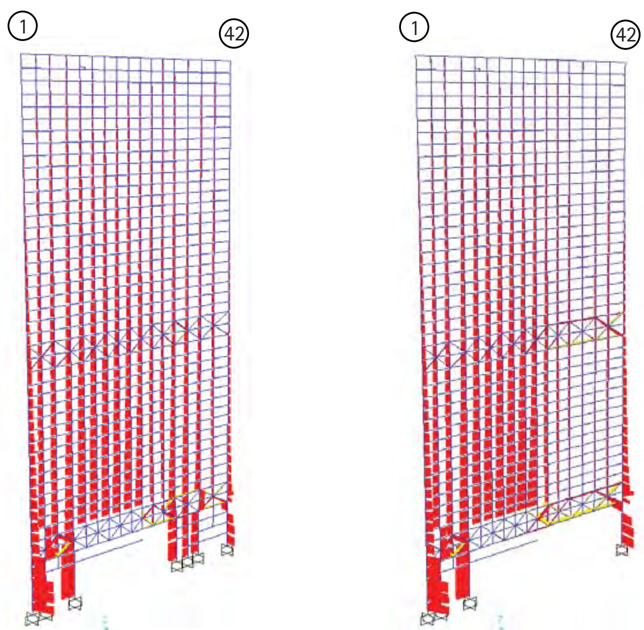


Figure 5.28 Redistribution of North exterior frame axial loads in Stage 4 due to buckling of Columns 44, 45 and 46

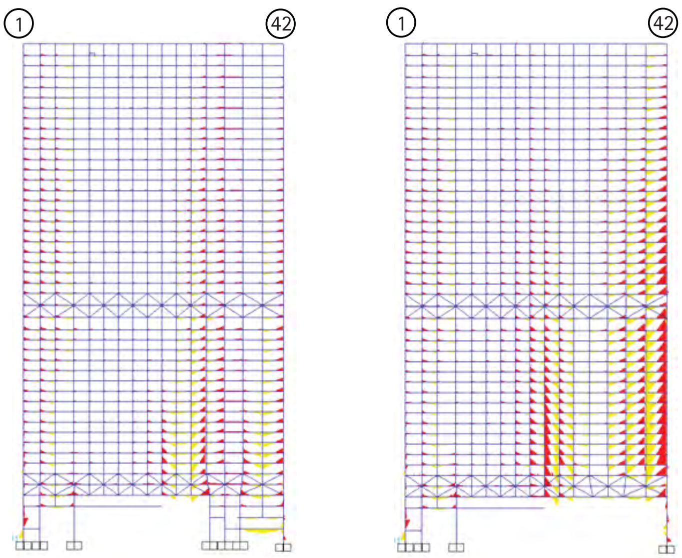


Figure 5.29 Increase in North exterior frame bending moments in Stage 4 due to buckling of Columns 44, 45 and 46

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.

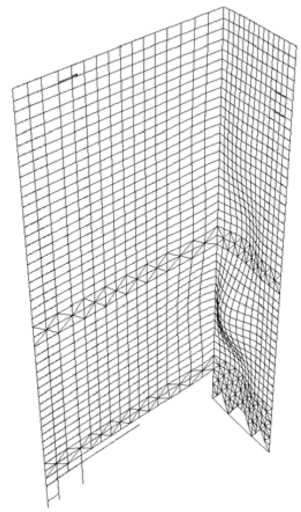


Figure 5.30 Buckled shape (factor=1.23) of east perimeter frame at Stage 4 in global model (partial view towards northeast)

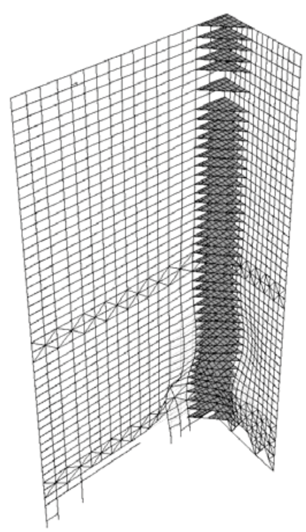


Figure 5.31 Buckled shape of a portion of a study model illustrating possible influence of corner floor slabs on the buckling behavior of the northeast corner of the perimeter frame

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.

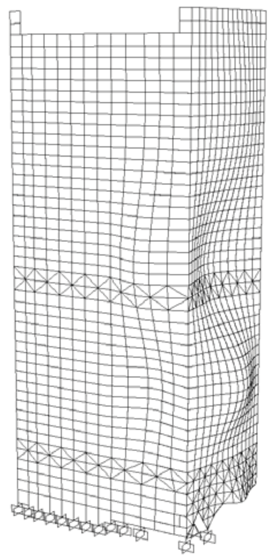
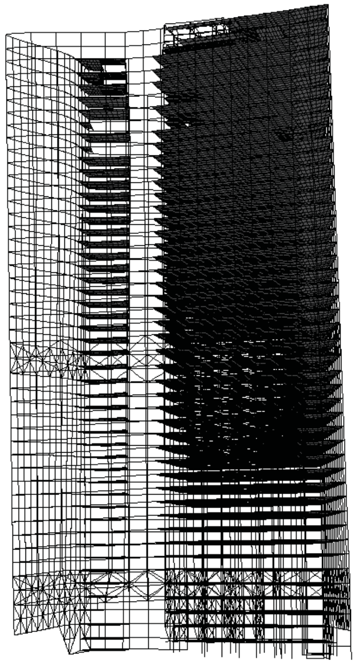


Figure 5.32 Buckled shape (factor=0.87) of east and south perimeter frame at Stage 5 in global model (partial view towards northwest)

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).



(a)

(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

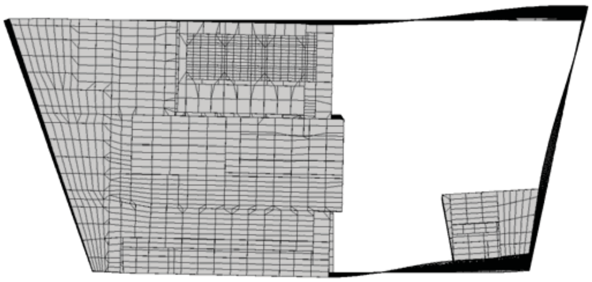


Figure 5.34 Deformed shape (plan, deformations not to scale) of global collapse model in Stage 5 following buckling of base of perimeter frame at the northeast corner

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.

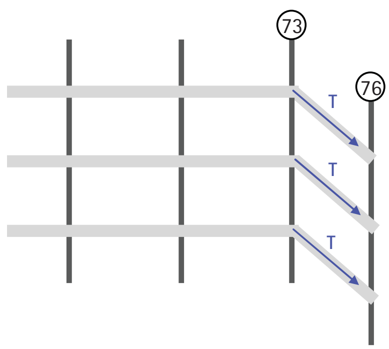


Figure 5.35 Basis for horizontal loading on western floor slabs

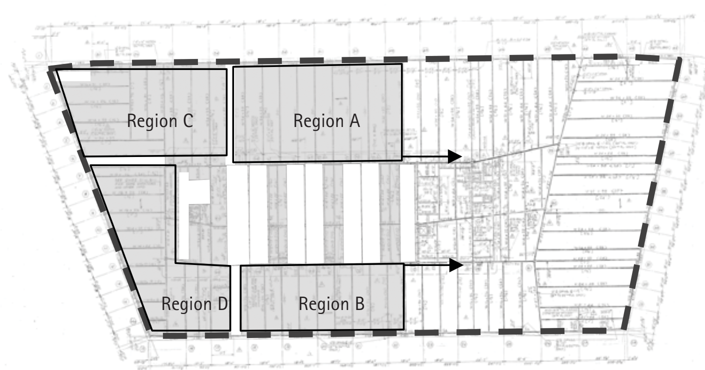


Figure 5.36 Horizontal loading on intact floor slab as floor areas tributary to Columns 76, 77 and 78 begin to collapse (perimeter moment frame shown with dashed line)

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).

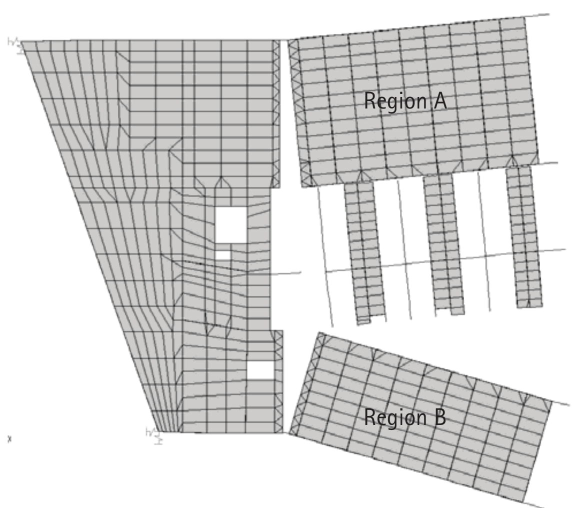


Figure 5.37 Rupture of trench headers from lateral loading leading to instability of Regions A and B of the diaphragm (deformations not to scale, failed links hidden from view for clarity)

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.

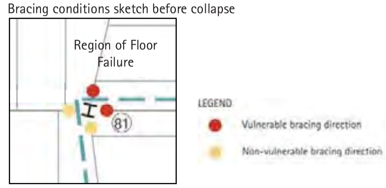
Figure 5.38 Input and Output for Column 81 Stability Analysis Illustrating Column Buckling Under Sustained Gravity Loads

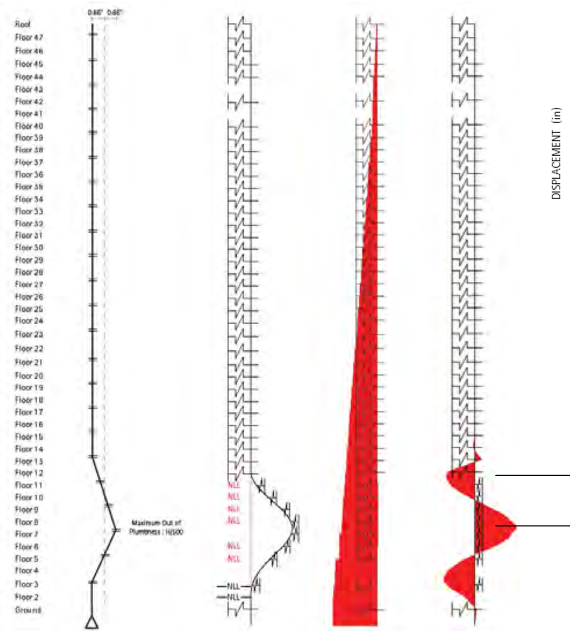
Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	SUSTAINED AXIAL COLUMN LOADS [kip]	FRAME SECTIONS	COLUMN BRACING PROPERTIES IN THE MODEL			
			North	South	East	West
Roof	-75.2	W14x90	-	-	LS	-
Floor 47	-160.3	W14x99	-	-	LS	-
Floor 46	-258.6		-	-	LS	-
Floor 45	-379.9	W14x145	-	LS	LS	-
Floor 44	-505.3		-	LS	LS	-
Floor 43	-507.5	W14x176	-	-	-	-
Floor 42	-633.5		-	LS	LS	-
Floor 41	-636.0	W14x211	-	-	-	-
Floor 40	-758.2		-	LS	LS	-
Floor 39	-880.3	W14x257	-	LS	LS	-
Floor 38	-1002.9		-	LS	LS	-
Floor 37	-1125.5	W14x311	-	LS	LS	-
Floor 36	-1248.8		-	LS	LS	-
Floor 35	-1372.2	W14x370	-	LS	LS	-
Floor 34	-1496.4		-	LS	LS	-
Floor 33	-1620.6	W14x426	-	LS	LS	-
Floor 32	-1745.5		-	LS	LS	-
Floor 31	-1870.5	W14x455	-	LS	LS	-
Floor 30	-1995.9		-	LS	LS	-
Floor 29	-2121.4	W14x500	-	LS	LS	-
Floor 28	-2247.4		-	LS	LS	-
Floor 27	-2373.6	W14x550	-	LS	LS	-
Floor 26	-2500.4		-	LS	LS	-
Floor 25	-2627.1	W14x605	-	LS	LS	-
Floor 24	-2755.2		-	LS	LS	-
Floor 23	-2912.2	W14x665	-	LS	LS	-
Floor 22	-3070.2		-	LS	LS	-
Floor 21	-3228.2	W14x665	-	LS	LS	-
Floor 20	-3356.8		-	LS	LS	-
Floor 19	-3485.8	W14x730	-	LS	LS	-
Floor 18	-3614.6		-	LS	LS	-
Floor 17	-3743.5	W14x730	-	LS	LS	-
Floor 16	-3872.5		-	LS	LS	-
Floor 15	-4001.5	W14x730 + 2 WEB PI 12.5x1	-	LS	LS	-
Floor 14	-4131.7		-	LS	LS	-
Floor 13	-4262.0	W14x730 + 2 WEB PI 12.5x1	-	LS	LS	-
Floor 12	-4392.3		-	LS	LS	-
Floor 11	-4522.7	W14x730 + 2 WEB PI 12.5x2	-	LS	NLL (9.8kip)	-
Floor 10	-4654.5		-	LS	NLL (9.8kip)	-
Floor 9	-4786.3	W14x730 + 2 WEB PI 12.5x3	-	LS	NLL (9.8kip)	-
Floor 8	-4919.1		-	LS	NLL (9.8kip)	-
Floor 7	-5211.7	W14x730 + 2 WEB PI's 12.5x3.5	-	LS	-	-
Floor 6	-5223.7		-	LS	NLL (12.8kip)	-
Floor 5	-5768.0	W14x730 + 2 WEB PI's 12.5x6.5	-	LS	NLL (10.2kip)*	-
Floor 4	-5783.5		-	-	-	-
Floor 3	-5988.1	W14x730 + 2 WEB PI's 12.5x4.5	-	LS	NLL (25.5kip)	-
Floor 2	-6063.8		-	-	NLL (39.1kip)	-
Ground	-6063.8		-	LS	LS	-

* 24in Single angle connection

Bracing conditions sketch before collapse





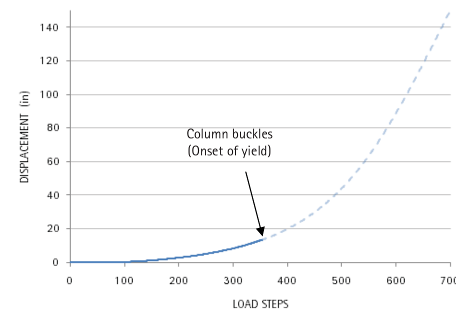
Initial geometry assumed per AISC Tolerances

Deformed Shape Under Sustained Loads

Axial Forces at Failure

Bending Moment (minor axis) at Failure

Displacement Plot at Floor 8 (minor axis)



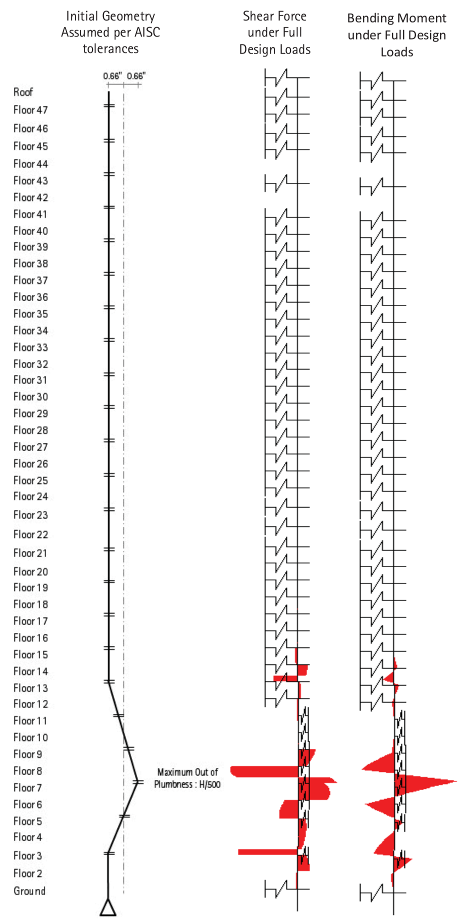
Cross Section: W14x730 + 2 Web Pl 12.5x1
Axial Force demand: 4361 kip
Major Axis Moment Demand: 12083 kip-in
Minor Axis Moment Demand: 12796 kip-in
Maximum Stress in Section: 51.2 ksi $< F_{ye}$ = 59.9 ksi (In W-Section)

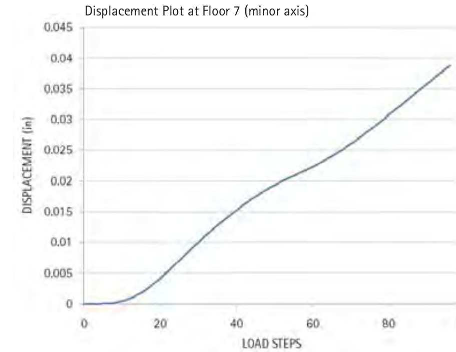
Cross Section: W14x730 + 2 Web Pl 12.5x3
Axial Force demand: 4884 kip
Major Axis Moment Demand: 16513 kip-in
Minor Axis Moment Demand: 18941 kip-in
Governing Stress in Section: 60.0 ksi $> F_{ye}$ = 59.9 ksi (In W-Section)

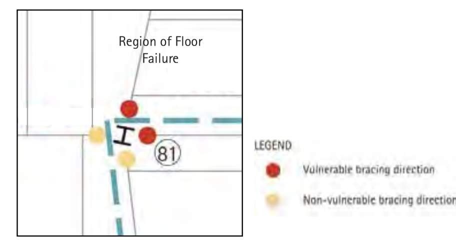
Figure 5.39 Input and output for stability analysis for Column 81 with 2% bracing capacity illustrating column stability under sustained gravity loads

Bracing Conditions, Sectional Properties and Loading in the model (LS= Linear spring, NLL=Nonlinear link)

FLOORS	FRAME SECTIONS	DESIGN AXIAL COLUMN LOADS [kip]	REQ CONN AXIAL CAPACITY (2%)	COLUMN BRACING PROPERTIES			
				North/South Direction	Link Force under Full Design Load	East/West Direction	Link Force under Full Design Load
Roof	W14x90	-87.0	1.74	-	-	LS	-
Floor 47	W14x99	-176.2	3.52	-	-	LS	-
Floor 46		-277.4	5.55	-	-	LS	-
Floor 45	W14x145	-404.8	8.10	LS	-	LS	-
Floor 44		-536.4	10.73	LS	-	LS	-
Floor 43	W14x176		0.00	-	-	-	-
Floor 42		-670.8	13.42	LS	-	LS	-
Floor 41	W14x211		0.00	-	-	-	-
Floor 40		-801.5	16.03	LS	-	LS	-
Floor 39	W14x257	-929.6	18.59	LS	-	LS	-
Floor 38		-1058.2	21.16	LS	-	LS	-
Floor 37	W14x311	-1186.9	23.74	LS	-	LS	-
Floor 36		-1316.2	26.32	LS	-	LS	-
Floor 35	W14x370	-1445.6	28.91	LS	-	LS	-
Floor 34		-1575.8	31.52	LS	-	LS	-
Floor 33	W14x426	-1706.0	34.12	LS	-	LS	-
Floor 32		-1837.0	36.74	LS	-	LS	-
Floor 31	W14x455	-1968.0	39.36	LS	-	LS	-
Floor 30		-2099.4	41.99	LS	-	LS	-
Floor 29	W14x500	-2230.9	44.62	LS	-	LS	-
Floor 28		-2363.0	47.26	LS	-	LS	-
Floor 27	W14x550	-2495.1	49.90	LS	-	LS	-
Floor 26		-2628.0	52.56	LS	-	LS	-
Floor 25	W14x605	-2760.8	55.22	LS	-	LS	-
Floor 24		-2894.8	57.90	LS	-	LS	-
Floor 23	W14x665	-3060.9	61.22	LS	-	LS	-
Floor 22		-3227.9	64.56	LS	-	LS	-
Floor 21	W14x665	-3394.9	67.90	LS	-	LS	-
Floor 20		-3529.6	70.59	LS	-	LS	-
Floor 19	W14x730	-3664.6	73.29	LS	-	LS	-
Floor 18		-3799.5	75.99	LS	-	LS	-
Floor 17	W14x730	-3934.4	78.69	LS	-	LS	-
Floor 16		-4069.4	81.39	LS	-	LS	-
Floor 15	W14x730 + 2 WEB PI 12.5x1	-4204.5	84.09	LS	-	LS	-
Floor 14		-4340.7	86.81	LS	-	LS	-
Floor 13	W14x730 + 2 WEB PI 12.5x1	-4477.1	89.54	LS	-	LS	-
Floor 12		-4613.4	92.27	LS	-	LS	-
Floor 11	W14x730 + 2 WEB PI 12.5x2	-4749.9	95.00	LS	-	NLL	Compression
Floor 10		-4887.7	97.75	LS	-	NLL	Compression
Floor 9	W14x730 + 2 WEB PI 12.5x3	-5025.6	100.51	LS	-	NLL	Compression
Floor 8		-5164.5	103.29	LS	-	NLL	16.51
Floor 7	W14x730 + 2 WEB PI's 12.5x3.5	-5462.8	109.26	LS	-	-	-
Floor 6		-5474.8	109.50	LS	-	NLL	10.78
Floor 5		-6040.6	120.81	LS	-	NLL	Compression
Floor 4	W14x730 + 2 WEB PI's 12.5x6.5	-6056.1	121.12	-	-	-	-
Floor 3		-6269.9	125.40	LS	-	NLL	Compression
Floor 2	W14x730 + 2 WEB PI's 12.5x4.5	-6350.7	127.01	-	-	NLL	2.56
Ground		-	-	LS	-	LS	-







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

COLUMN DOES NOT BUCKLE

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).

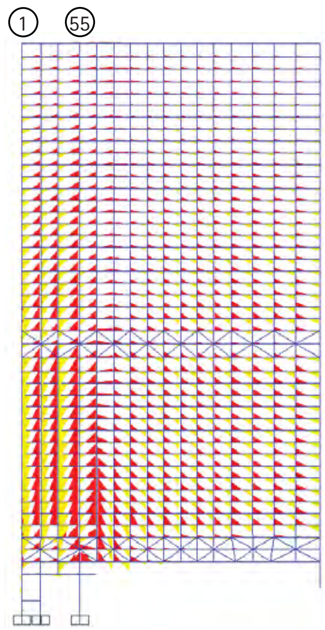


Figure 5.40 Bending moment diagram of north façade in Stage 6 indicating perimeter frame failures in the region of Column 55

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

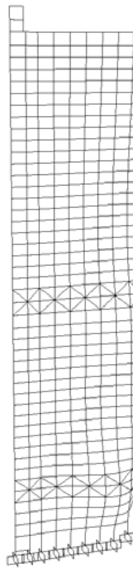


Figure 5.42 Buckled shape of south perimeter frame in Stage 6 of the global Model (looking northeast)

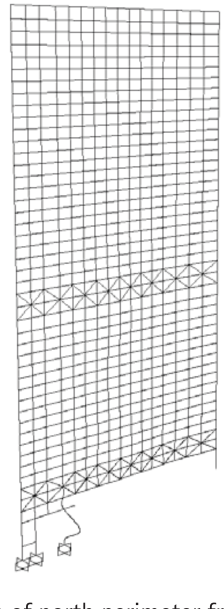


Figure 5.43 Buckled shape of north perimeter frame in Stage 6 of the global Model (looking northeast)

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*, 3rd Edition, CRC

6 Anderson T, *Fracture Mechanics: Fundamentals and Applications*, 3rd 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

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*

APPENDIX A

WTC7 Knife Connection Report
By Anthony Ingraffea PE PhD

WTC7 Knife Connection Study Report

Submitted to

Mark Antin Esq.

Gennett, Kallman, Antin and Robinson, P.C

6 Campus Drive

Parsippany, NJ 07054

by

A. R. Ingraffea, Ph.D., P.E.

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.

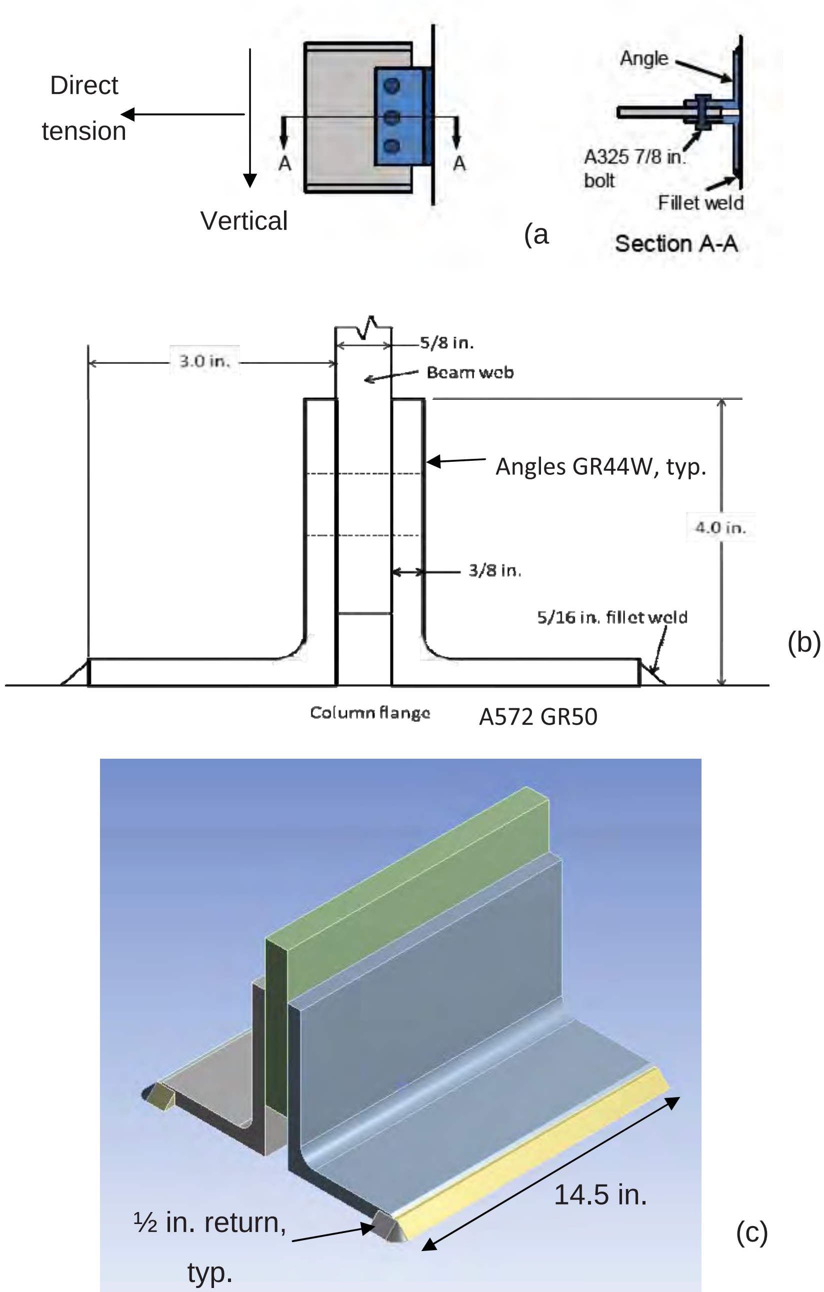


Figure 1. (a) Schematic of typical beam-column knife connection. (b) Column 79 knife connection in WTC7, cross-section. (c) Column 79 knife connection in WTC7, perspective (dimensions based on Frankel Steel Limited Drawing No. 1091, Rev May 23 1985)

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_u, 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]."

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

where

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

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.

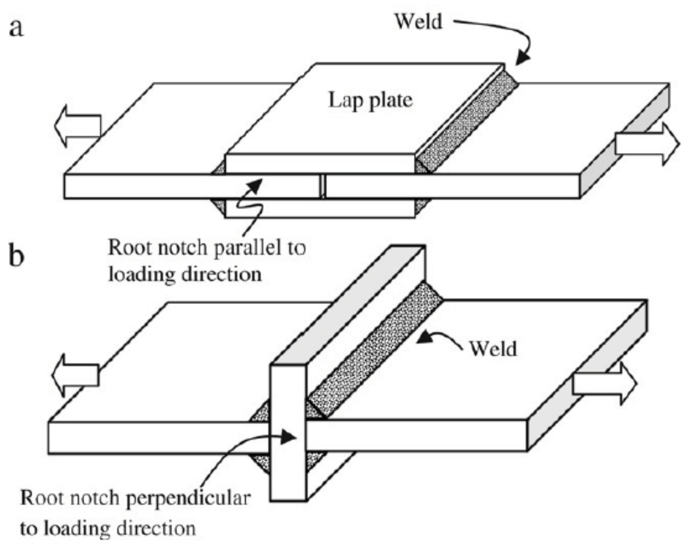


Fig. 1. Transverse welds in (a) Lap-welded specimens and (b) Cruciform type specimens.

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

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

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:

$$1 \geq (V/V_u)^2 + (P/P_u)^2 \qquad (2)$$

where

V = applied shear load/inch of weld

V_u = 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_I. 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, elasto-plastic stress fields in the connection, especially in the weld area, Δ/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, Δ/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_y = 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_u = 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.

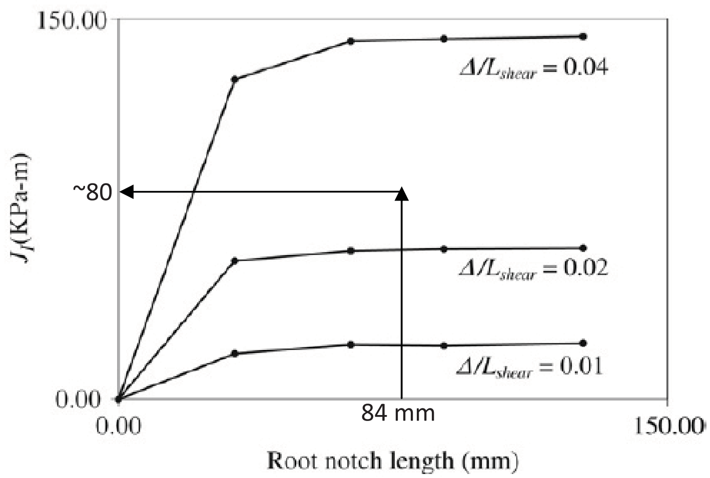


Fig. 13. Schematic plot showing the dependence of the toughness demand (J-integral) on root notch length for various deformations.

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
	8	145	0.21
E70T7-K2	12	406	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 (J)		
		F_y[a] (MPa)	F_u[b] (MPa)	d_o/d_f	ε[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**

[a] Measured yield stress, based on 0.2% offset method; static value.
[b] Measured ultimate strength; static value.
[c] $\varepsilon = \ln(d_o/d_f)^2$ = average true strain across necked cross section of tension coupon.

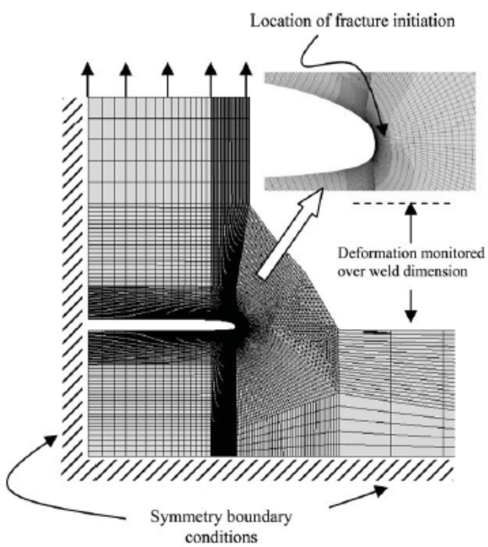


Fig. 12. Deformed finite element mesh of cruciform specimen, with inset figure showing a magnified view of the equivalent plastic strain contours in the notch tip region.

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

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_{Ic} 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_y. The contours in Figure 7b show that the effective stress is substantially above uniaxial yield, 77 ksi here, and above F_u 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, δ_t). A well-known, approximate relationship, based on empirical testing and finite element analysis, between J and δ_t is

$$J = M \times F_y \times \delta_t \tag{3}$$

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. δ_t relationship shown in Figure 9, with a δ_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.*

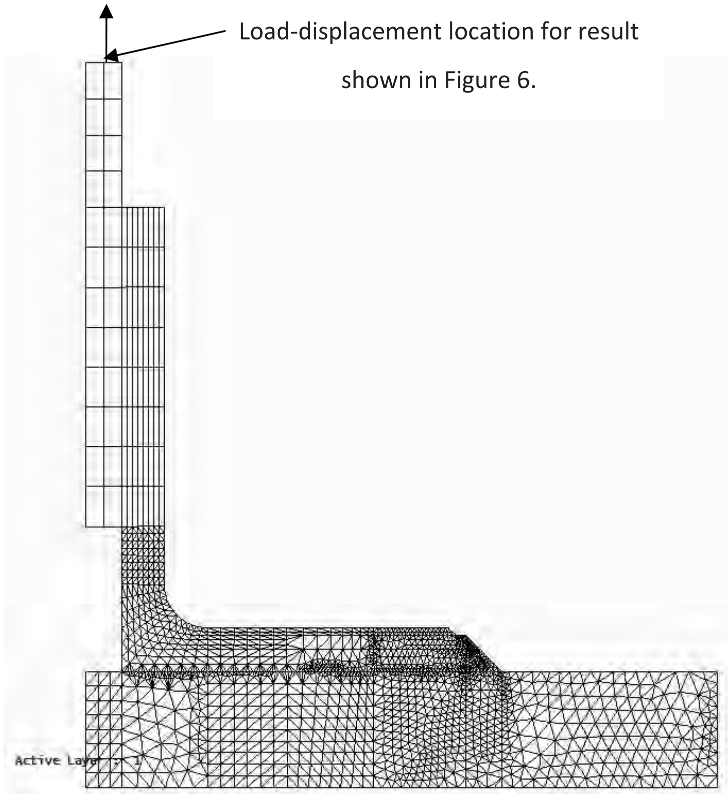


Figure 5a. Un-deformed FRANC2D mesh of WTC7 knife connection using symmetry.

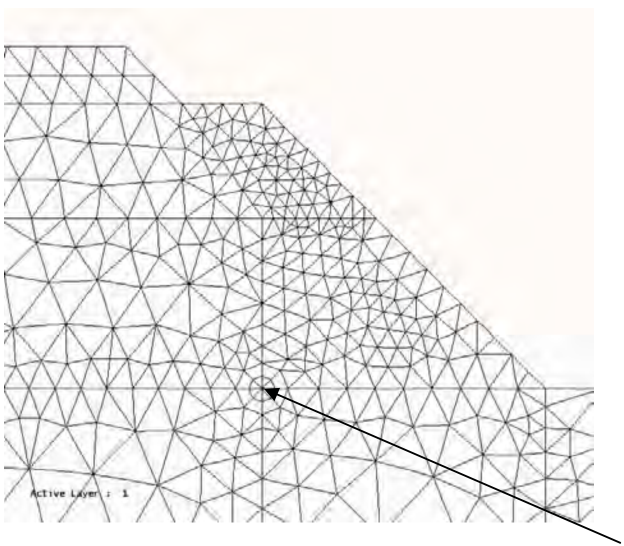


Figure 5b. Detail of mesh in Figure 5a around weld and crack front.

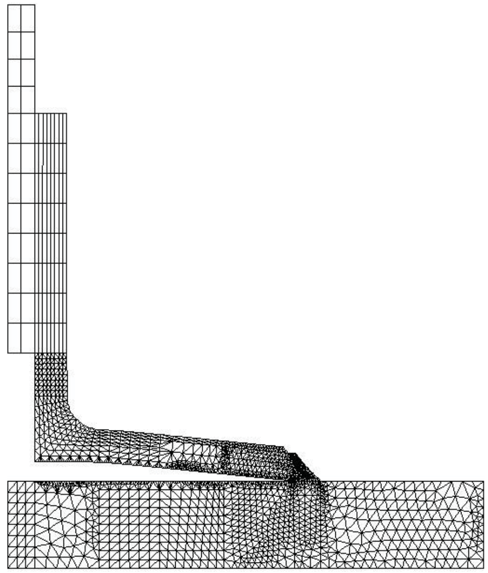


Figure 5c. FRANC2D-predicted deformation at load = 3.25 kips. Prying action on the fillet weld evident. No amplification.

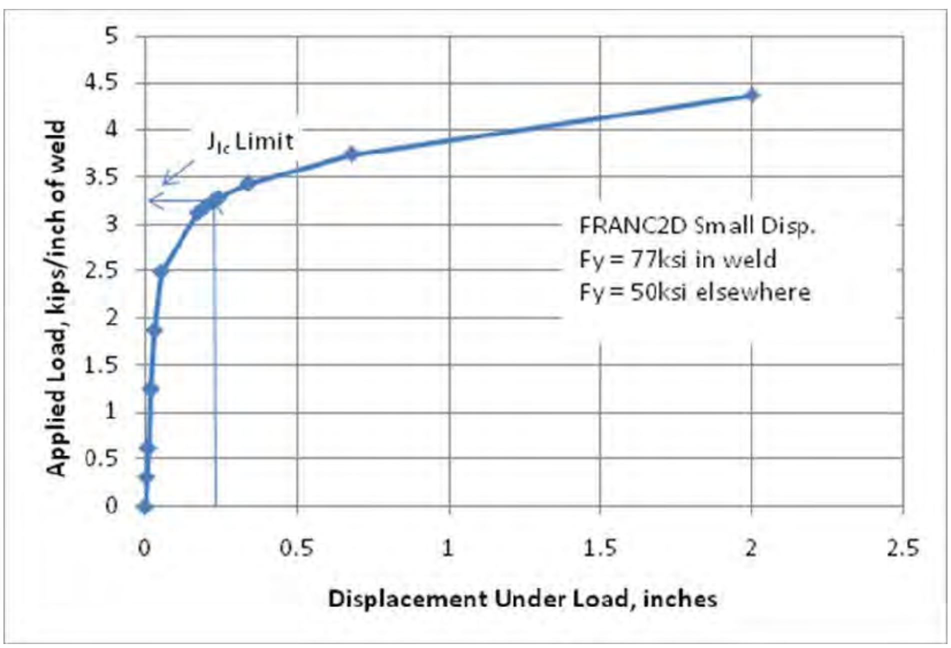


Figure 6. Predicted load-displacement relationship from FRANC2D, small displacement theory. The J_{IC} limit shown is for a suspected upper-bound value of 0.83 kips/in obtained by Kanvinde (2009) for E70T7 weld filler. No shear force included.

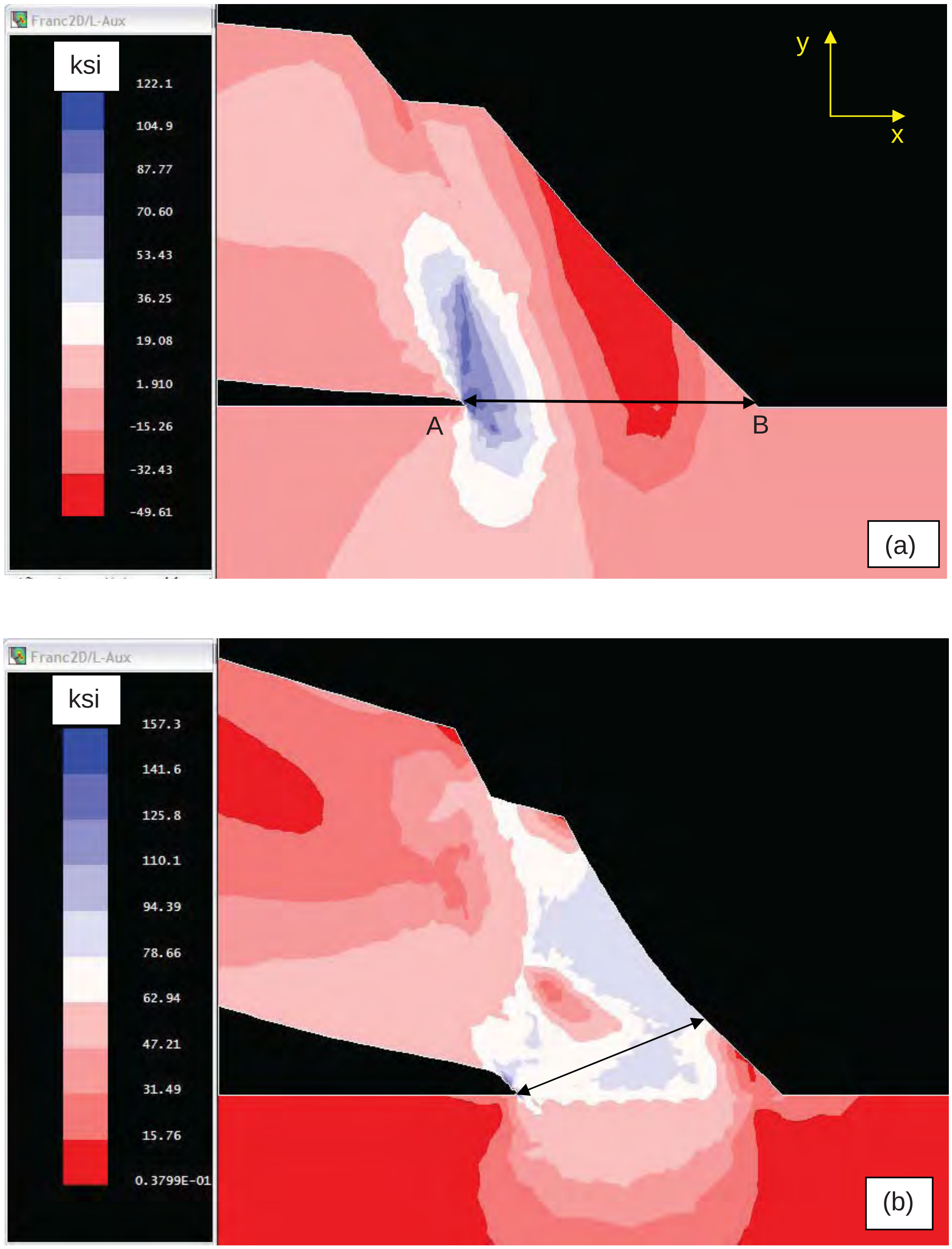


Figure 7. (a) FRANC2D-predicted y-stress contours at a load of 3.25 kips/inch of weld; (b) effective stress at a load of 3.75 kips/inch of weld on deformed shape (magnification = 1).

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_u. *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).

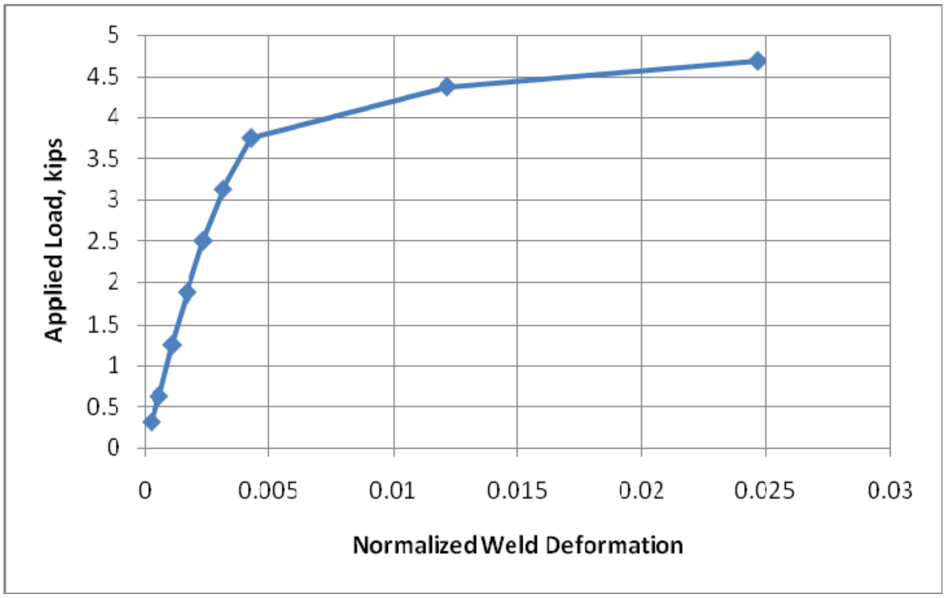


Figure 8. Predicted load versus normalized weld deformation, Δ/L_{shear}.

From FRANC2D with small displacement theory.

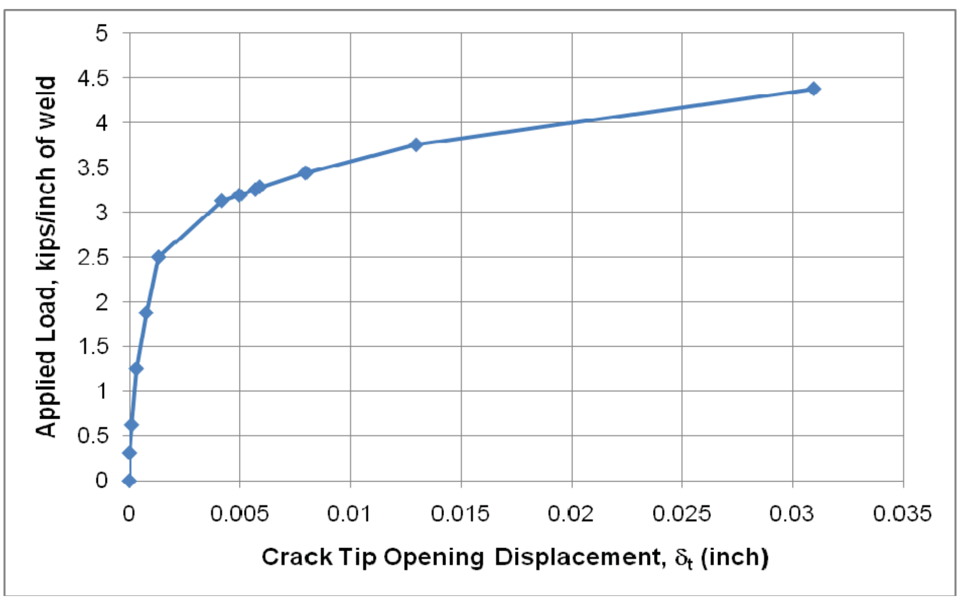


Figure 9. Predicted load versus crack tip opening displacement, δ_t.

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 δ_t.

Figure 12 shows the ANSYS-predicted load-displacement plot. At a load level of 4.7 kips/inch of weld, ANSYS predicts a J_I = 0.83 kips/in (145 kPa m), the upper-bound critical value, J_{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,

$$K^2_{Ic} = 5*E*CVN \quad \text{(psi sqrt(in), psi, ft-lb)} \tag{4}$$

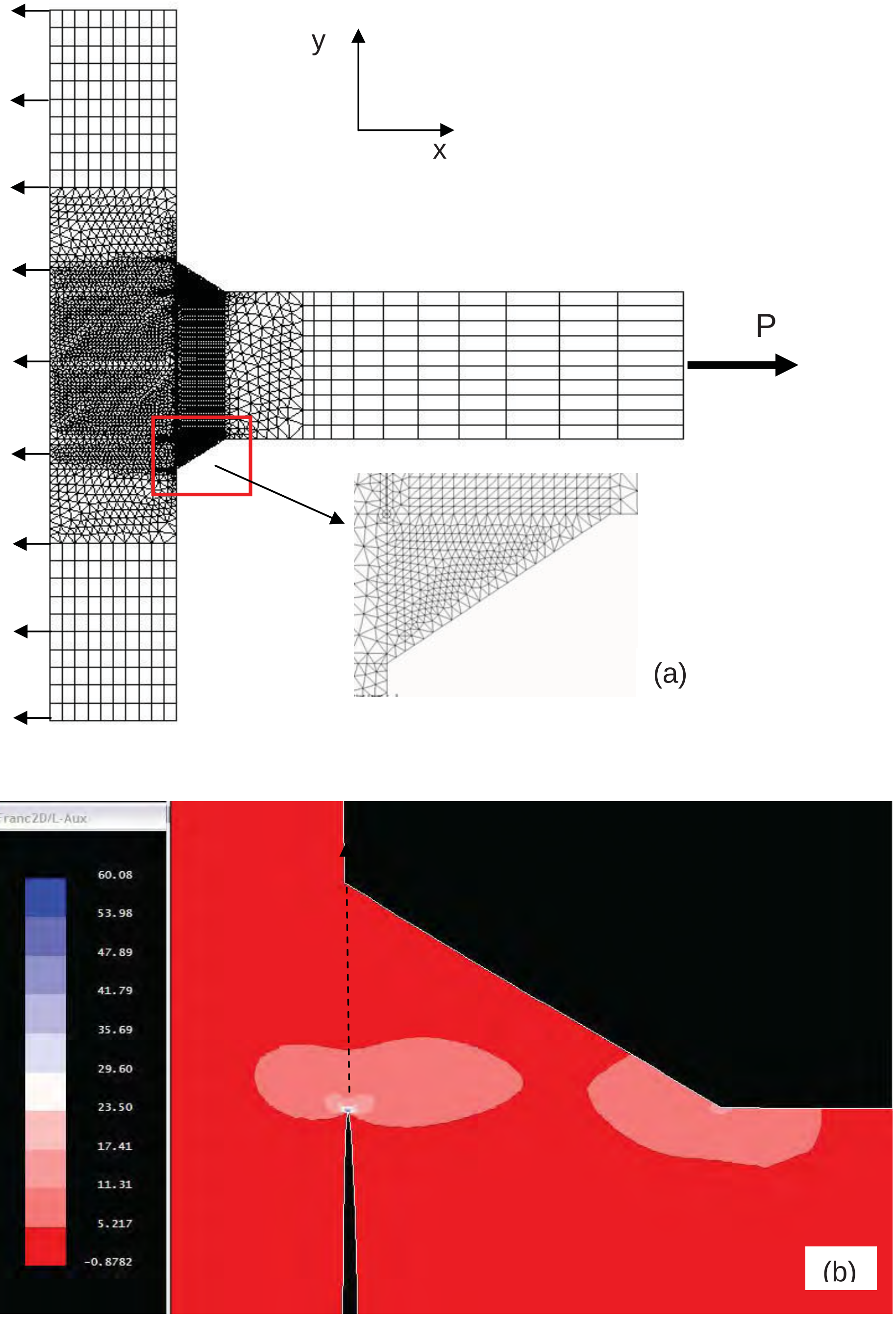
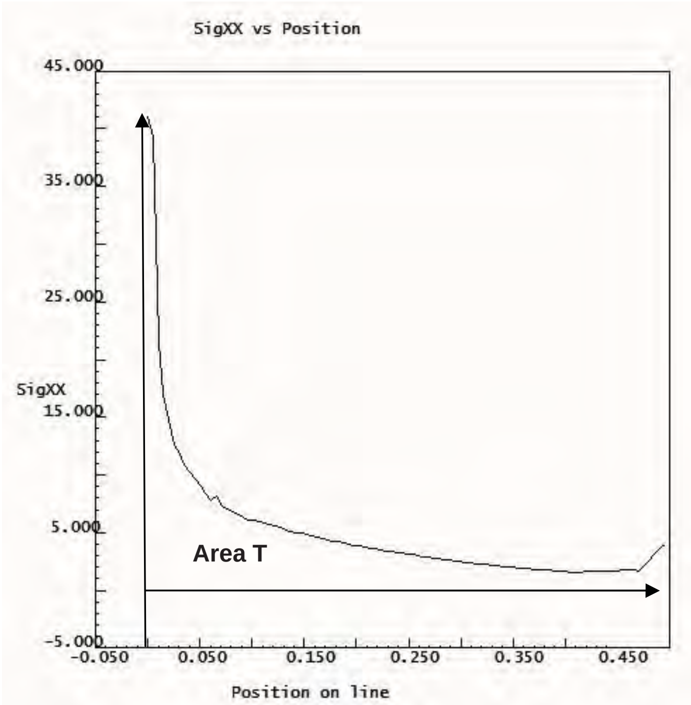


Figure 10. (a) FRANC2D model of cruciform connection tested and analyzed by Kanvinde (2009). (b) Contours of x-stress in weld region at a load, P, of 5kips/inch of weld. *Displacement magnification factor is 100.*



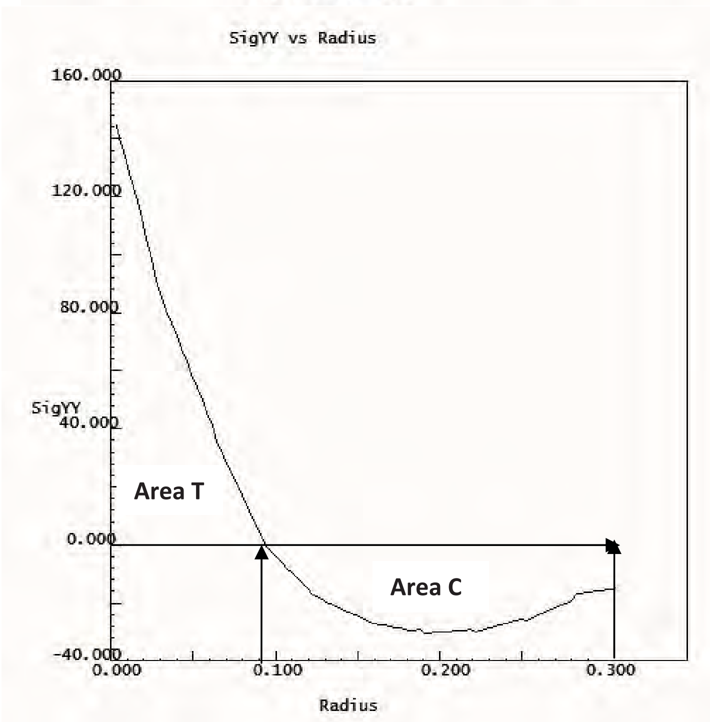


Figure 11. (a) Distribution of x-stress along dashed line shown in Figure 10b, cruciform connection. (b) Distribution of y-stress along line A-B shown in Figure 7a, knife connection.

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°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:

$$J_{Ic} = K^2_{Ic}\,(1-\nu^2)/\,E \qquad (5)$$

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,

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

Equation 6 is used for the conversion from measured CVN values in Table 4, because it has been observed that, at 21°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.

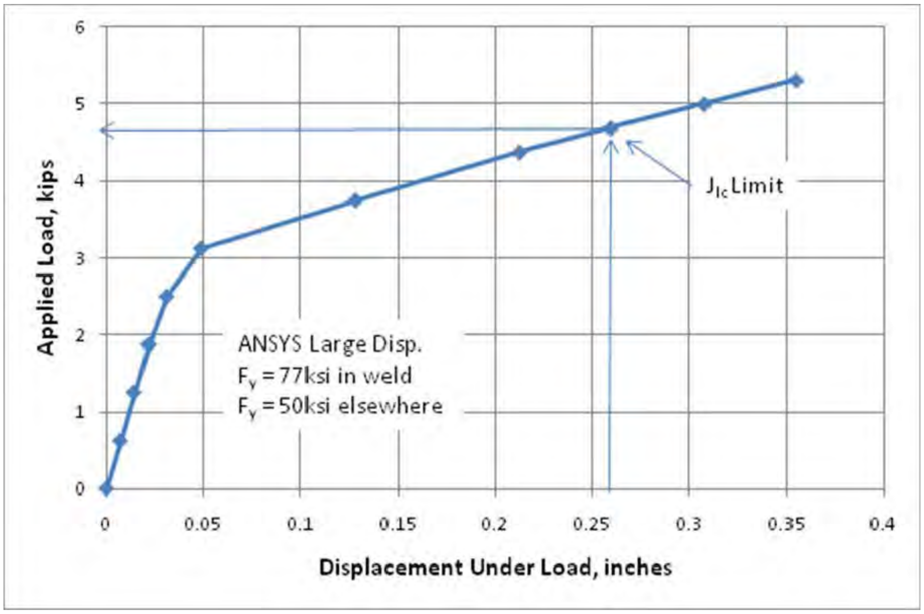


Figure 12. Predicted load-displacement relationship from ANSYS 2D with large displacement theory. The J_{IC} limit shown is for an upper-bound value of 0.83 kips/in. No shear force included.

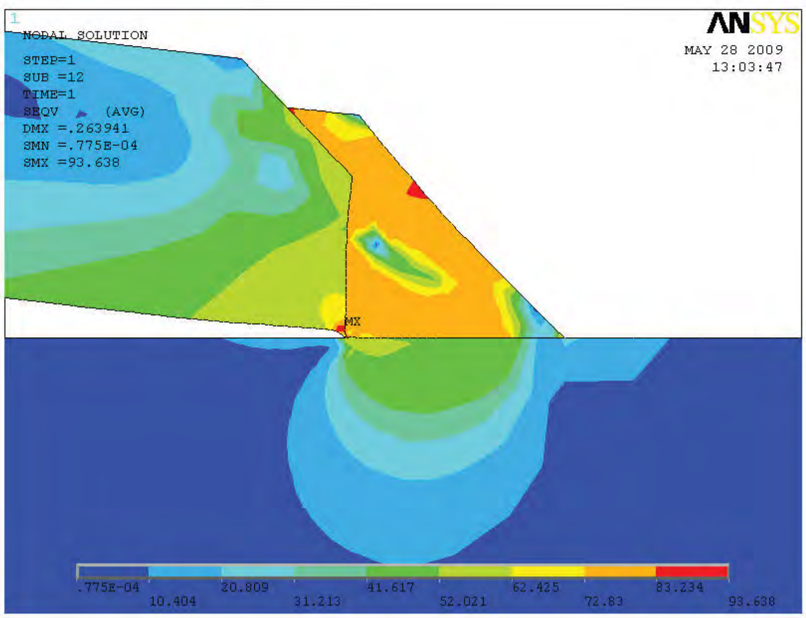


Figure 13. 2D ANSYS-predicted effective stress contours (ksi), on deformed shape (magnification = 1) at a load of 4.7 kips/inch of weld, large deformation theory.

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

Electrode	CVN Min, J (ft-lb) @21 C	CVN Max, J (ft-lb) @21 C	K_{Ic} Min[a] MPa $m^{1/2}$ (ksi in ^{½})	K_{Ic} Max[a] MPa $m^{1/2}$ (ksi in ^{½})	J_{Ic} Min[b] kPa m (k/in)	J_{Ic} Max[b] kPa m (k/in)	Source
E70T-4	8 (6)	26 (19)	31.9 (29)	57.1 (52)	4.8 (0.027)	15.1 (0.086)	Ojdrovic, 1997
E70T-4 (avg)	NA	18.6 (13.7)	NA	49.5 (45)	NA	10.9 (0.062)	Civjan, 2000
E70T-4	4 (2.9)	29 (21.3)	23.1 (21)	61.5 (56)	2.3 (0.013)	17 (0.097)	Fisher, 1996

[a] using equation 4

[b] using equation 5

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.

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

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

[c] calibrated using finite element calculations and physical tests

[d] using equation 6

[e] using calibrated J_{Ic} values and equation 5

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, 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.

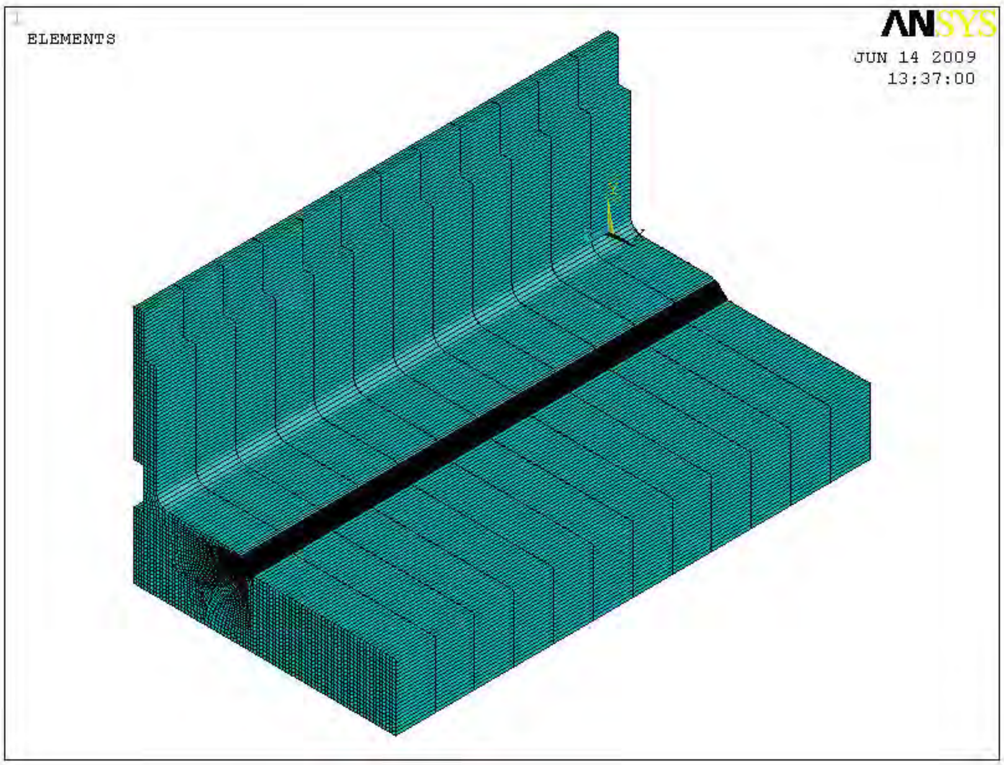


Figure 13. 3D ANSYS FE model without weld returns used to calculate effect of the returns and of vertical shear on J_I values.

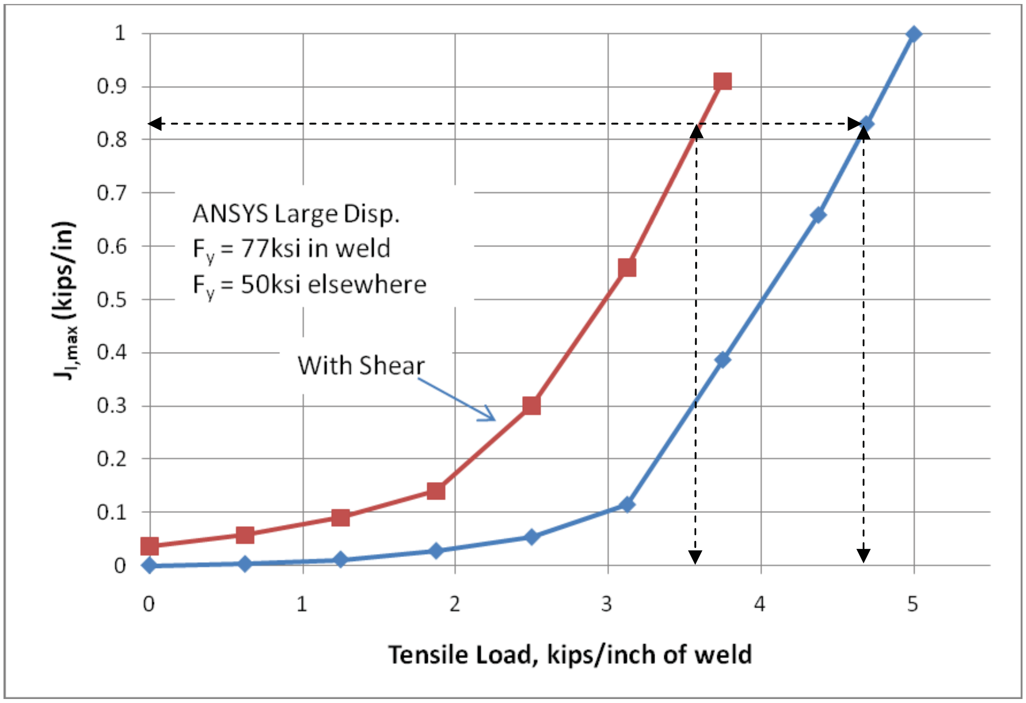


Figure 14. Comparison between maximum J_I values in the weld, with and without vertical shear force on the connection, no returns. Detail at low values of $J_{I,max}$ shown in Figure 15.

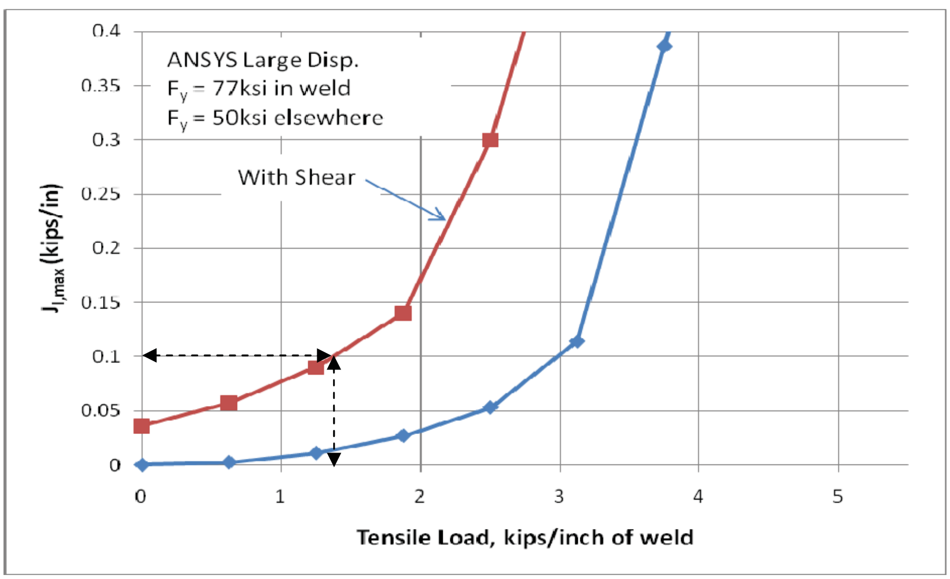
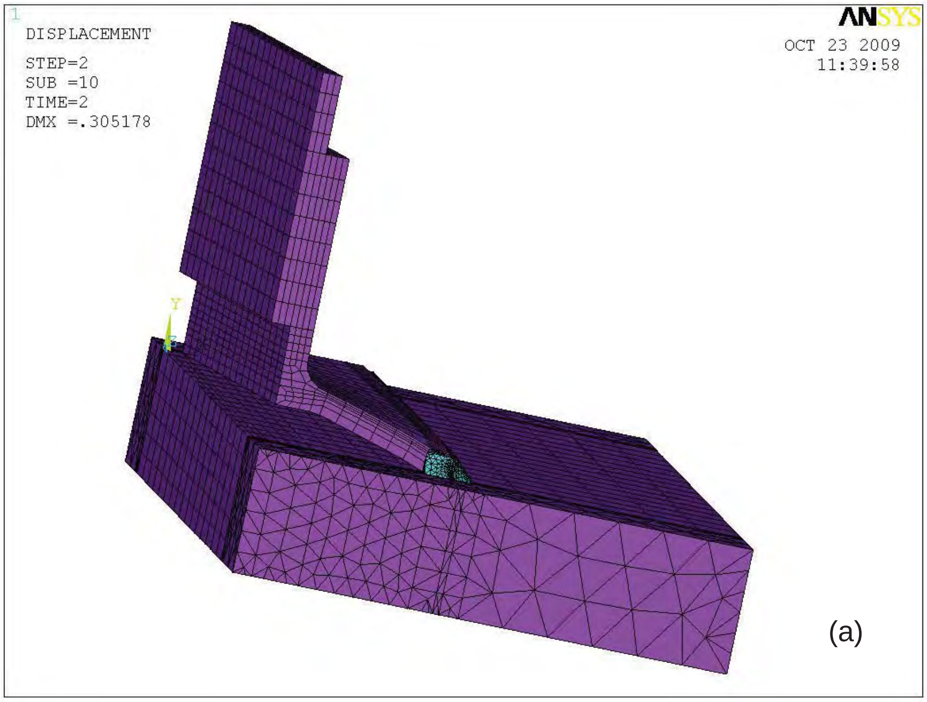


Figure 15. Detail of data shown in Figure 14. Dashed lines indicate capacity at a toughness of 0.10 kips/inch.



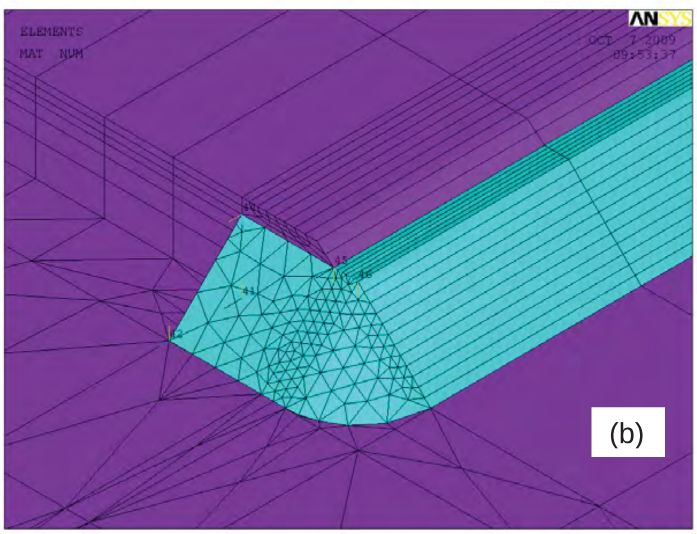


Figure 16. (a) Displaced shape of ANSYS model with weld returns at a tensile load of 3.75 kips/inch of weld, with shear, and a magnification factor of 2, view looking from top to bottom of connection. (b) Detail of model around a weld return.

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.*

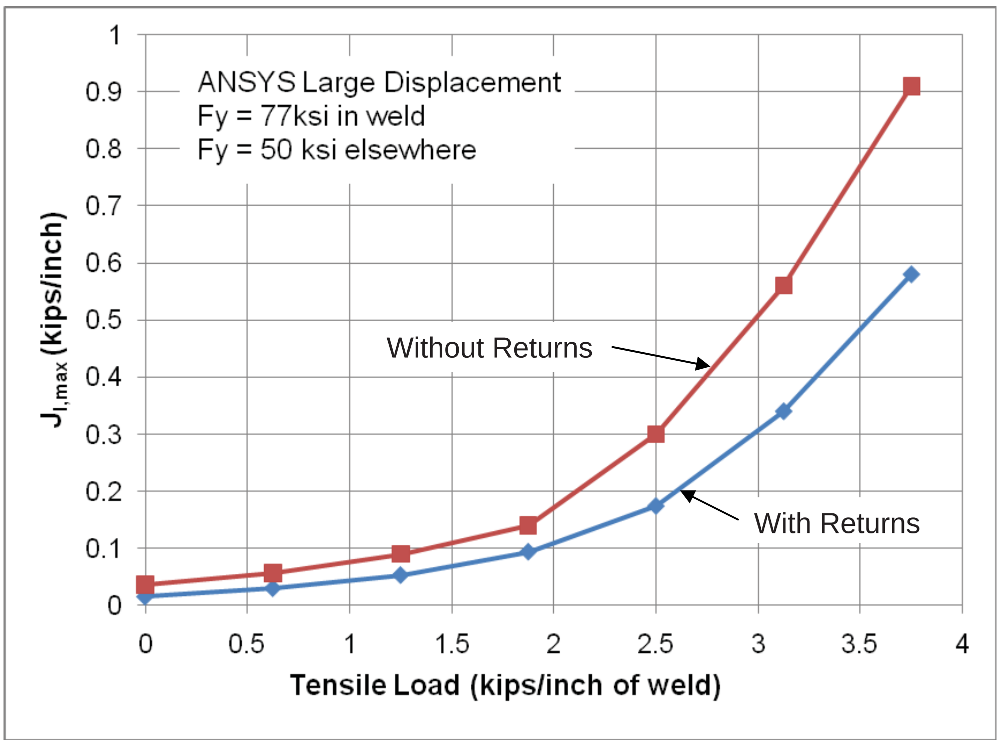


Figure 17. Maximum J_I along weld versus tensile load, including shear, with and without and weld returns. Detail at low values of $J_{I,max}$ shown in Figure 18.

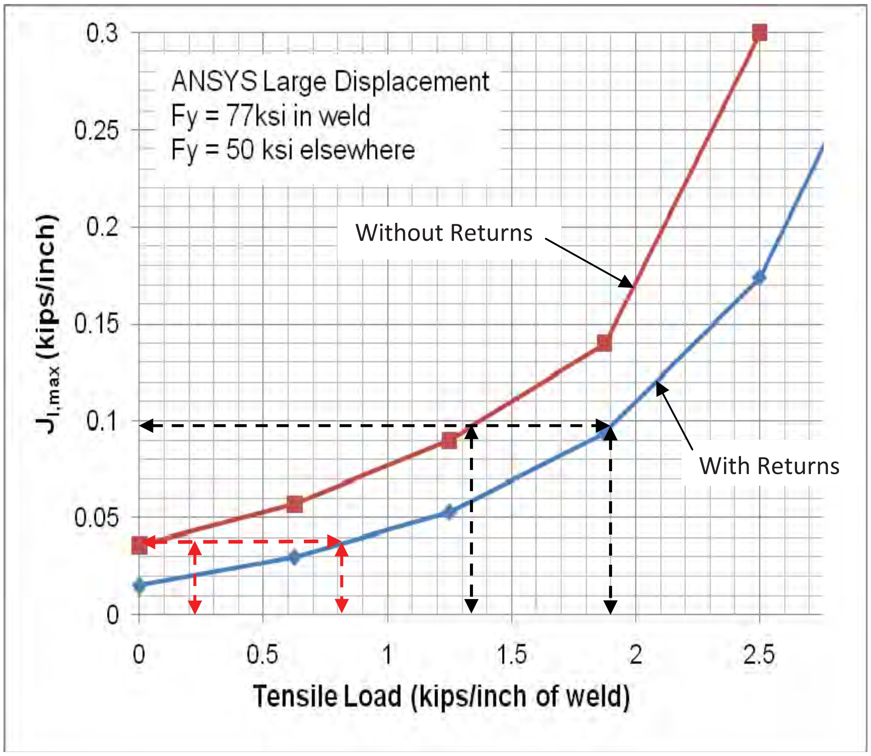


Figure 18. Maximum J_I along weld versus tensile load, including shear, with and without and weld returns. Detail at low values of $J_{I,max}$. Dotted lines indicate capacity at highest measured toughness of pre-Northridge E70-T4 weld material, black, and at median value (0.038 kip/in), red, shown in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

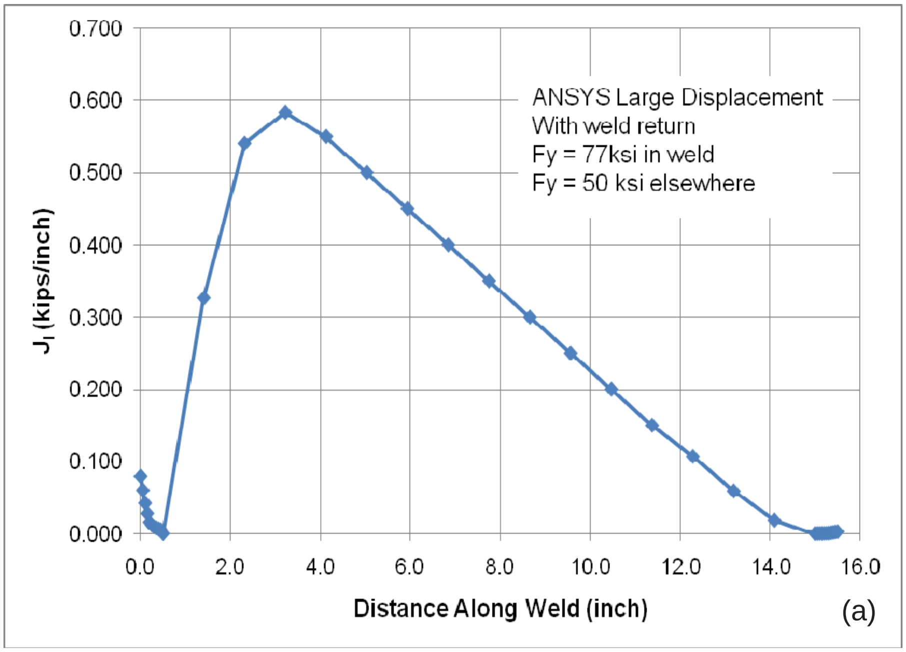
5.0 SUMMARY

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

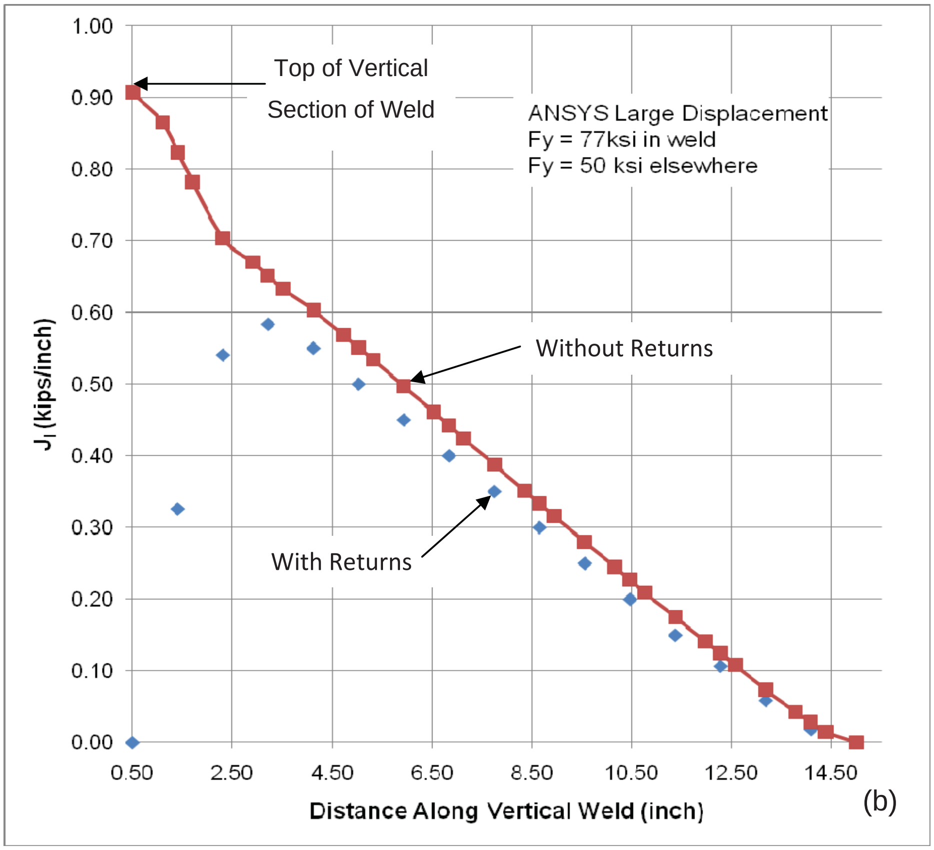


Figure 19. (a) Predicted distribution of J_I along entire length of weld with weld returns, for tensile load of 3.75 kips/inch of weld. (b) Comparison of distribution of J_I along length of vertical section of weld with and without weld returns, for tensile load of 3.75 kips/inch of weld.

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_{Ic}, 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_{IC} 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_u 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:
  - 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, 2nd 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.

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.

4. 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, 2nd 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.

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.

APPENDIX B

Floor Collapse Analysis Report

Guy Nordenson and Associates

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

225 Varick Street 6th Flr
New York NY 10014 USA
Tel 212 766 9119
Fax 212 766 9016
www.nordenson.com

TABLE OF CONTENTS

		Page
B1.0	**INTRODUCTION**	**B1**
B1.1	Description of Progressive Floor Collapse	B1
B1.2	Description of Floor Collapse Analysis Approach	B1
B2.0	**FLOOR COLLAPSE ASSUMED GEOMETRY**	**B4**
B2.1	SAP2000 Single Floor Model Analysis	B4
B2.2	Idealized Collapse Geometry	B7
B3.0	**FLOOR COLLAPSE ENERGY DISSIPATION**	**B11**
B3.1	Identification of Failure Modes and Energies	B11
B3.2	Composite Slab Tensile Fracture	B12
B3.3	Composite Slab Shear Fracture	B18
B3.4	Rotational Deformation at Slab Boundaries	B20
B3.5	Rotational Deformation at Slab Failure Yield Lines	B24
B3.6	Falling Girder Plastic Deformation at Impact	B26
B4.0	**FLOOR COLLAPSE POTENTIAL ENERGY**	**B28**
B4.1	SAP2000 Single Floor Model Analysis	B28
B4.2	Equivalent Collapsed Floor Section Geometry	B29
B4.3	Equivalent Collapsed Floor Section Potential Energy	B29
B5.0	**FLOOR IMPACT AND FAILURE**	**B30**
B5.1	Girder Impact Location	B30
B5.2	Girder Stiffness at Impact Location	B32
B5.3	Impact Force Transmission to Column Connections	B33
B5.4	Connection Failure	B34
B6.0	**VERTICAL COLLAPSE PROPAGATION**	**B36**
B6.1	Propagation Analysis Assumptions	B36
B6.2	Floor 13 to Ground Floor Collapse Propagation	B37

B7.0 CONCLUSIONS B39

B8.0 MOMENT ROTATION CURVES AND HINGE ENERGIES B40

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:

$$\left[\left(\begin{array}{l}\text{Potential Energy of}\\ \text{Falling Floor Slab}\end{array}\right) - \left(\begin{array}{l}\text{Energy Dissipated}\\ \text{in Failure of Floor}\end{array}\right)\right] \quad \text{VS} \quad \left(\begin{array}{l}\text{Energy Required to Fail}\\ \text{Girder Connection to}\\ \text{Column at Floor Below}\end{array}\right)$$

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).

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.

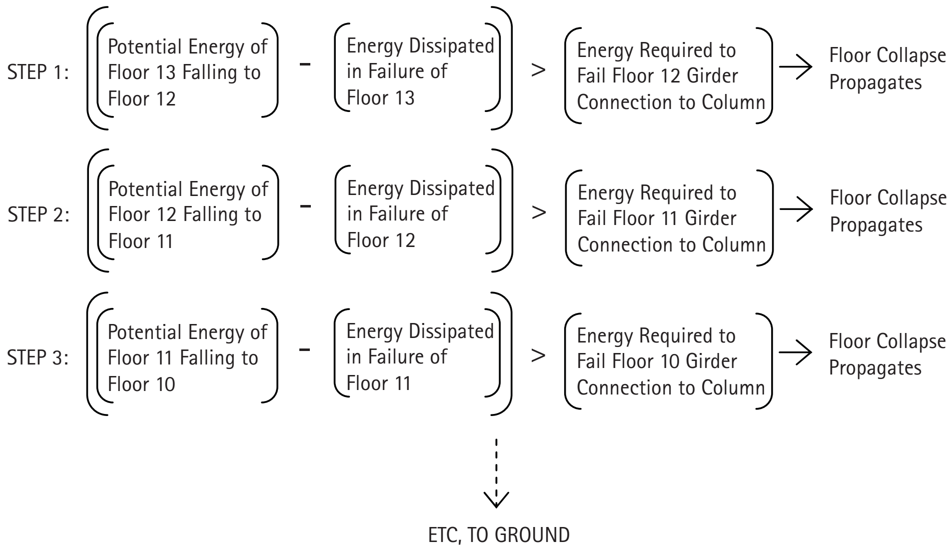


Figure B1.1 Conservative methodology for floor collapse assessment

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 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.

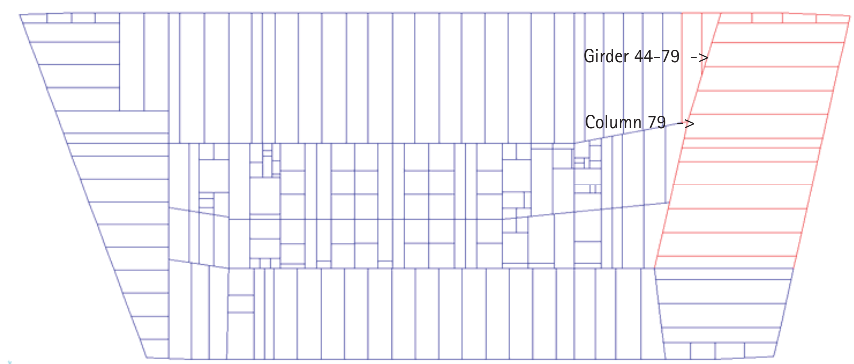


Figure B2.1 Level 13 partial floor SAP2000 model extents (geometry consistent with global model at Level 13)

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.

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.

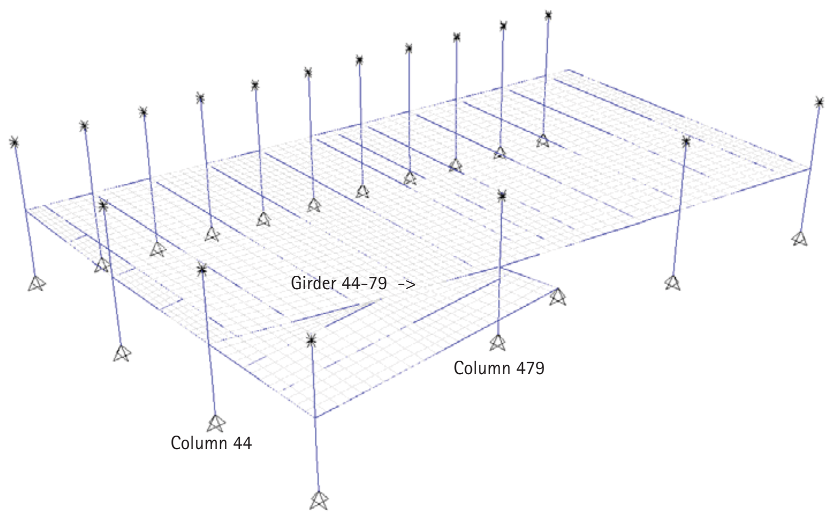


Figure B2.2 Undeformed SAP2000 model with Girder 44-79 connected to Column 79

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.

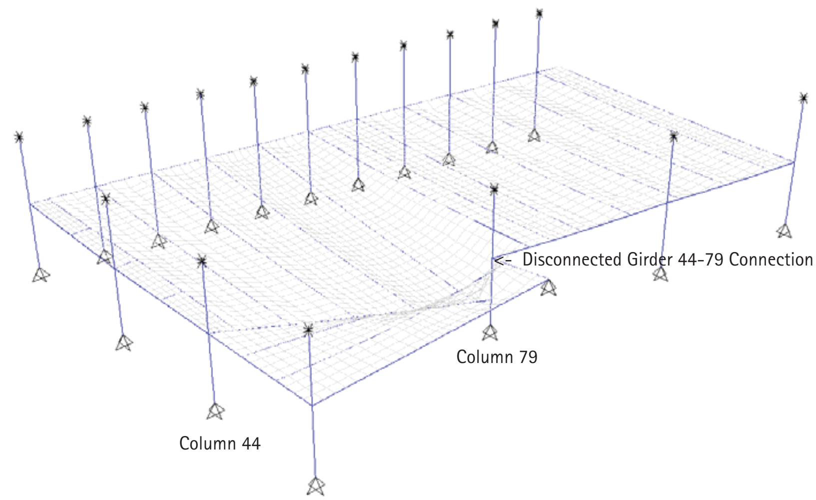


Figure B2.3 Deformed SAP2000 model under gravity loads with disconnected girder (4x amplified elastic deformation)

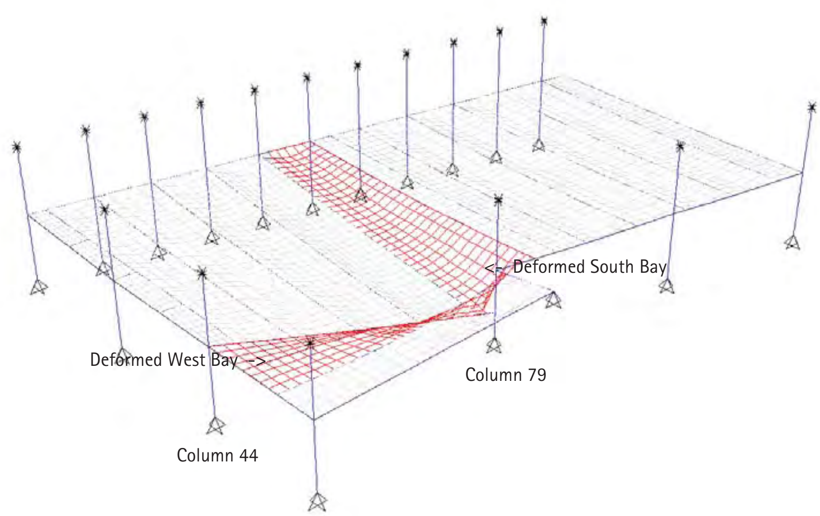


Figure B2.4 Deformed SAP2000 model under gravity loads with disconnected girder (4x amplified elastic deformation) (Note: effect of trench header at south bay not accounted for in approximated deformations)

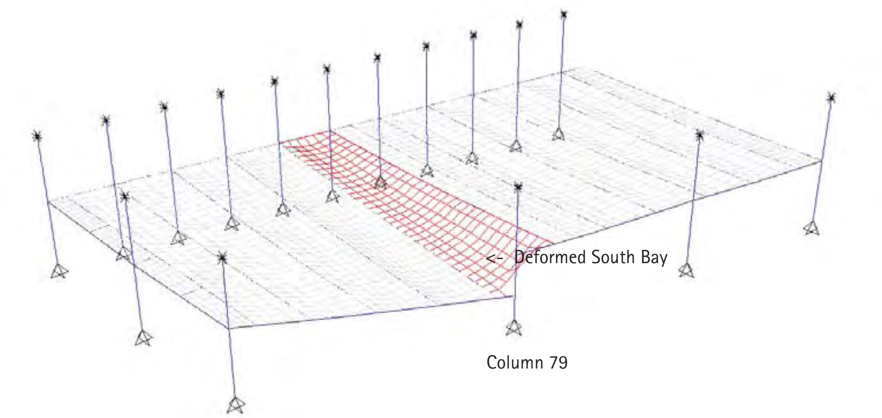


Figure B2.5 SAP2000 Model under gravity loads with disconnected girder (4x amplified elastic deformation; west slab sections removed from view for clarity) (Note: effect of trench header at south bay not accounted for in approximated deformations)

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.

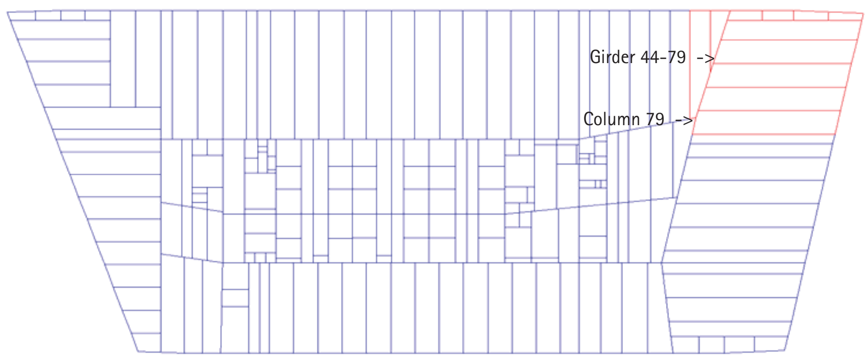


Figure B2.6 Level 13 partial floor extents of idealized geometry model

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.

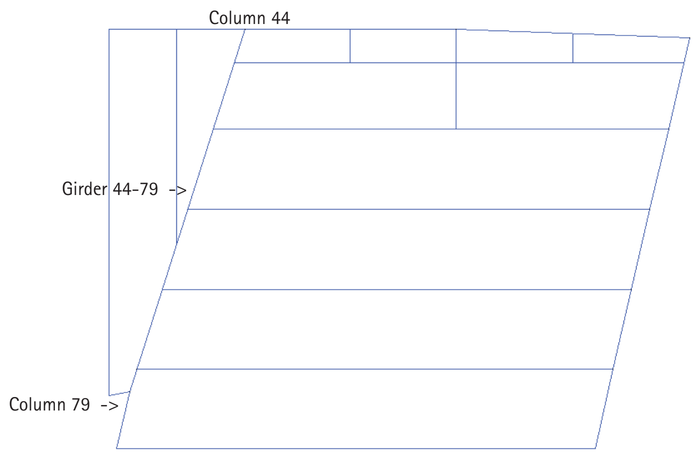


Figure B2.7 Undeformed idealized geometry model extents

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.

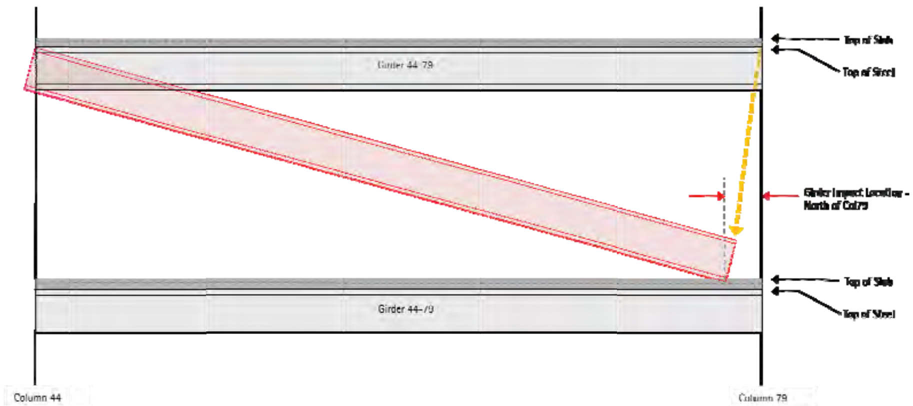


Figure B2.8 Girder 44-79 simplified rotation diagram

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.

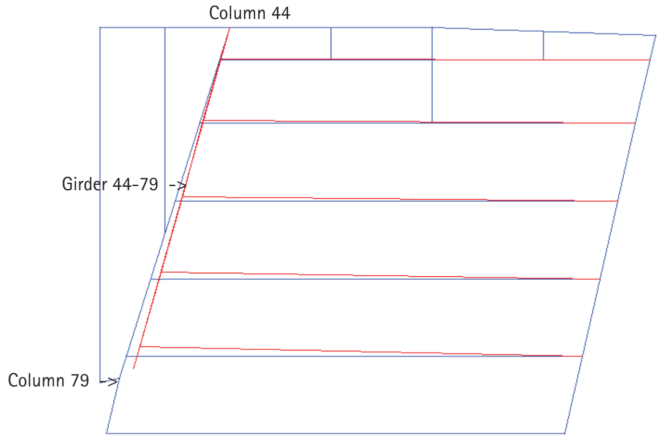


Figure B2.9 Projected plan of idealized deformation geometry (blue represents undeformed geometry, red represents deformed geometry)

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.

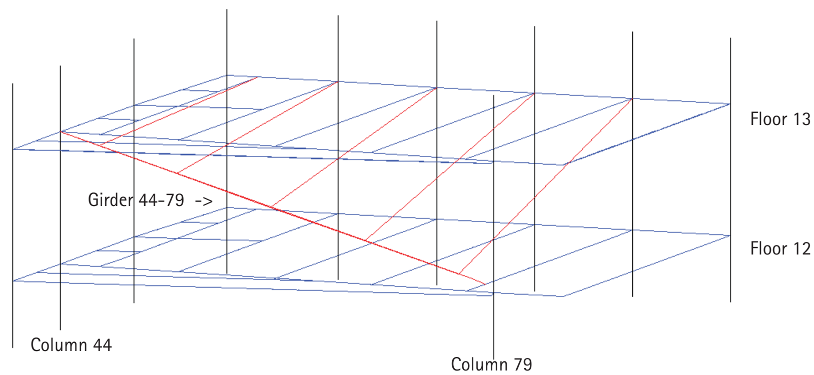


Figure B2.10 Perspective of idealized deformation geometry (blue represents undeformed geometry, red represents deformed geometry)

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.

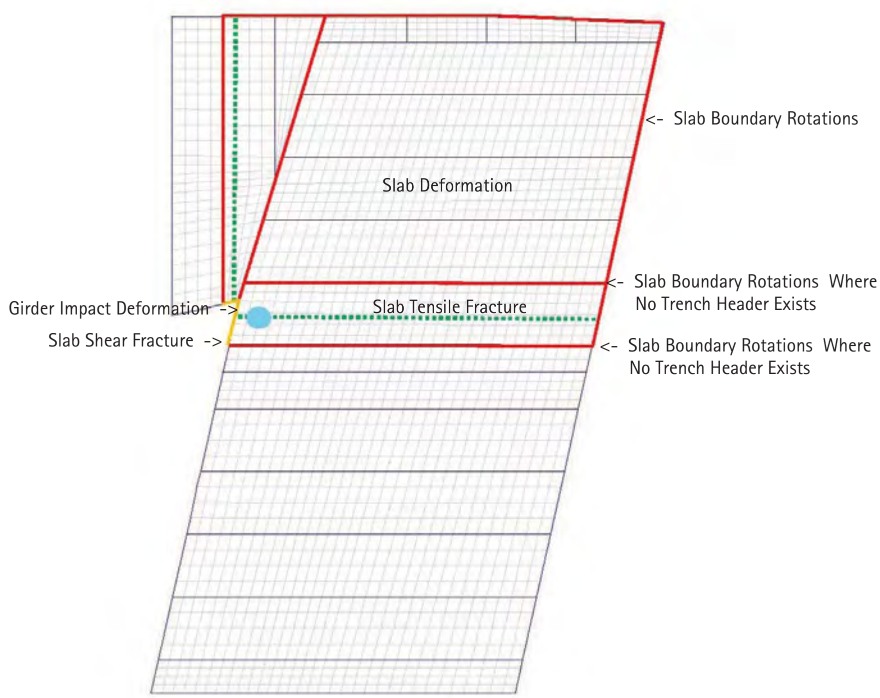


Figure B3.1 Energy dissipation modes accounted for in analysis

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.

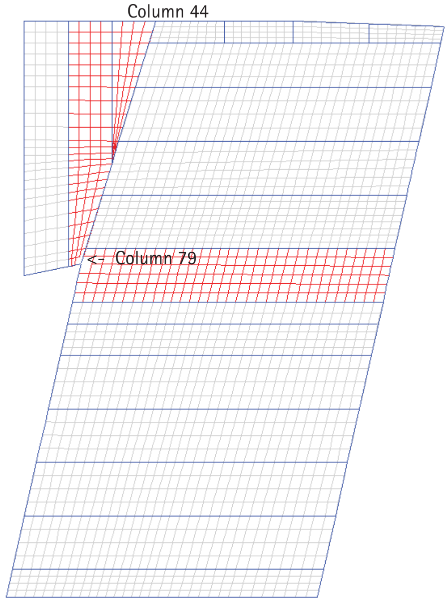


Figure B3.2 "Tensile Fractured" composite slab bays

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 [in]	Mom of Inertia [in4]	Deck Gauge	Deck Thickness [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
	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)

Floor	Direction	Top Slab Reinf Each Way [in2/ft]	Bott Slab Reinf Each Way [in2/ft]	Added Reinf Perp to Spandrels [in2/ft]	Top Cover [in]
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 deck)	Major	0.31 (#5@12)	-	0.2 (#4@12)	0.75*
	Minor	0.31 (#5@12)	-	0.2 (#4@12)	0.75*
	Angle	0.438	-	-	0.75*
7 (8" thick slab)	Both	0.72 (#7@10)	0.372 (#5@10)	-	0.75*
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.

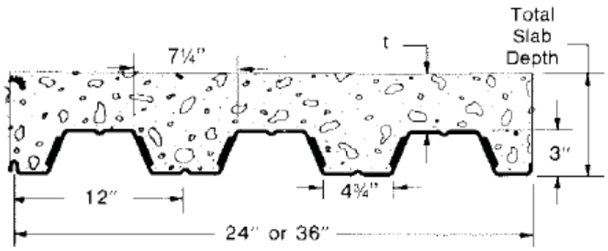


Figure B3.3 Metal deck profile diagram from the Steel Deck Institute

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 $4\text{x}10^{-4}$ kip-in/in^2 and the fracture energy of metal deck, wire mesh, and steel reinforcing bars (Gc) was taken as 0.5 kip-in/in^2 (Refs 6 and 10).

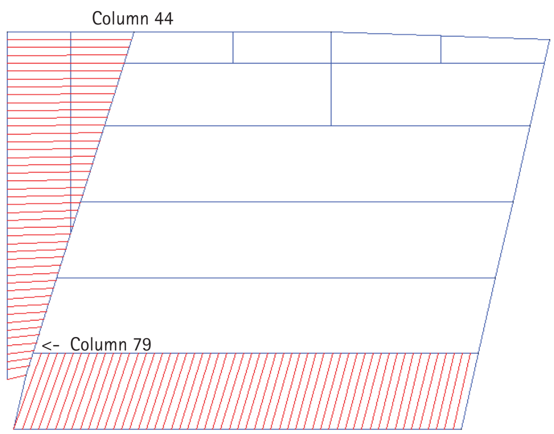


Figure B3.4 Tensile fractured slab section strips plan view

At Floor 13, each 1ft-wide section of composite slab consists of $48in^2$ of concrete, $0.572in^2$ of metal deck, and 0.028 in^2 of wire mesh. Based on the fracture energies noted above, each strip of slab is therefore capable of dissipating energy equal to:

$$\begin{aligned}
&(48in^2 \times 4x10^{-4} \text{ kip-in/in}^2)\\
&+ (0.572in^2 \times 0.5 \text{ kip-in/in}^2)\\
&+ (0.028 \text{ in}^2 \times 0.5 \text{ kip-in/in}^2)\\
&= 0.3 \text{ kip-in}
\end{aligned}$$

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

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

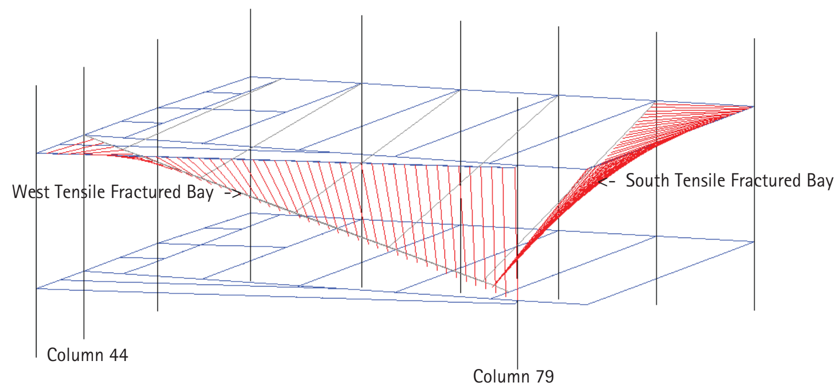


Figure B3.5 Tensile fractured slab section strips perspective view

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)

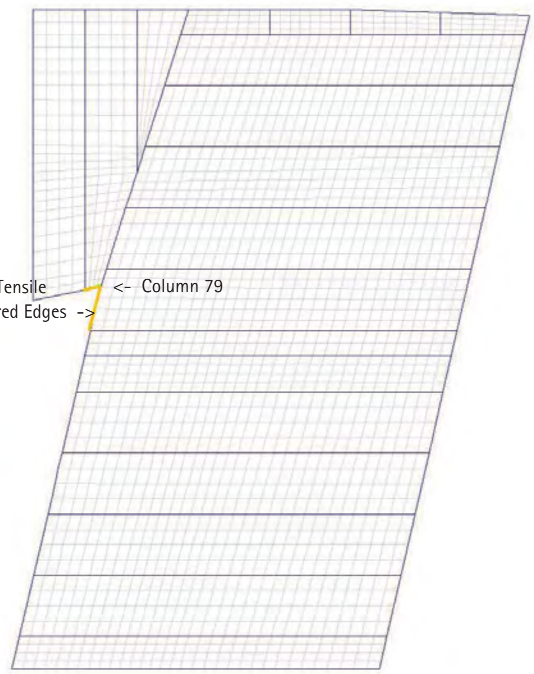


Figure B3.6 Shear/tensile fractured slab boundaries

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 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.

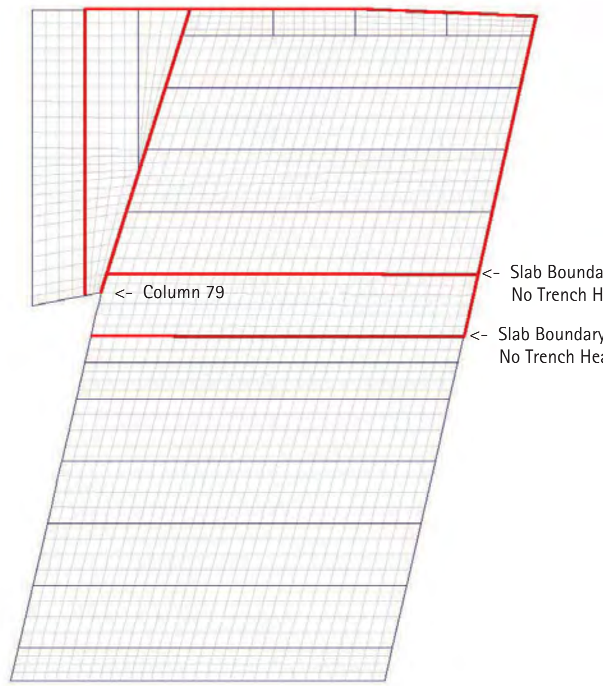


Figure B3.7 Slab boundary rotational deformation

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.

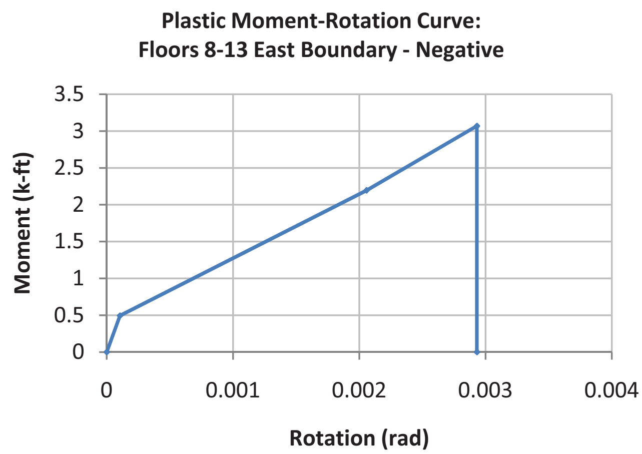


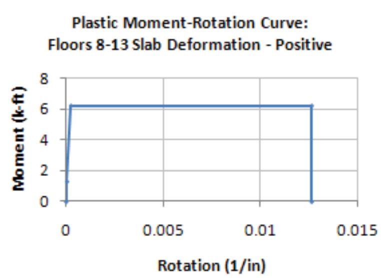
Figure B3.8 Typical composite slab plastic moment-rotation curve

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.

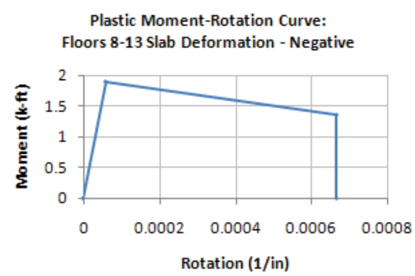
A common approximation for the plastic hinge length of depth/2 was used. The area under each moment-rotation curve was calculated, representing the maximum energy dissipated by the rotation of a 1 foot-wide strip of slab. The slab boundaries were conservatively assumed to have undergone full plastic 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.

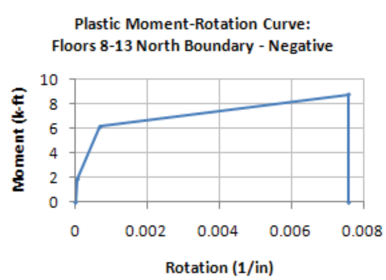
Slab Deformation
Positive



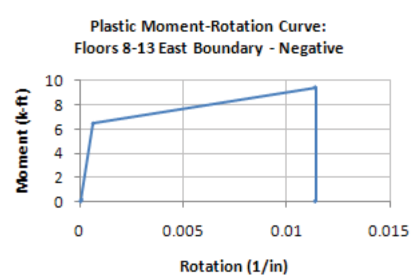
Slab Deformation
Negative



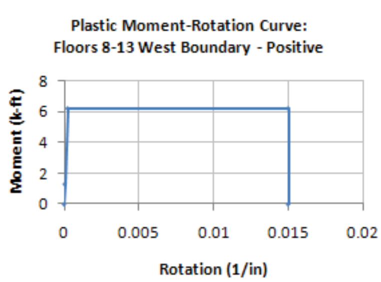
North Boundary
Negative



East Boundary
Negative



West Boundary
Positiv



West Boundary
Negat

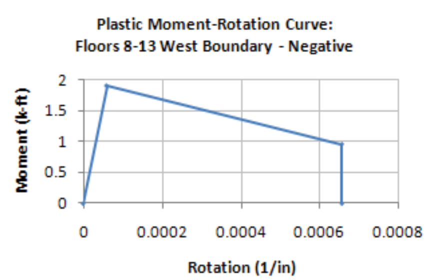


Figure B3.9 Levels 8-13 Composite slab plastic moment-rotation curves

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.

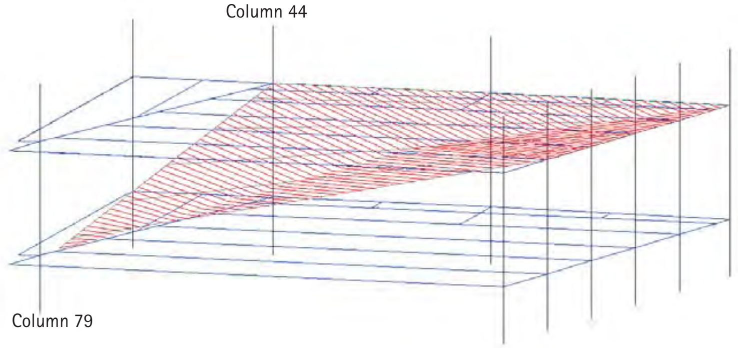


Figure B3.10 Perspective view of floor deformation hinge strips

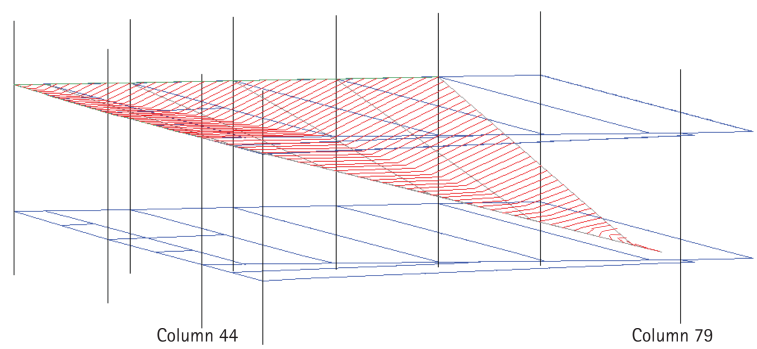


Figure B3.11 Alternate perspective view of floor deformation hinge strips

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.

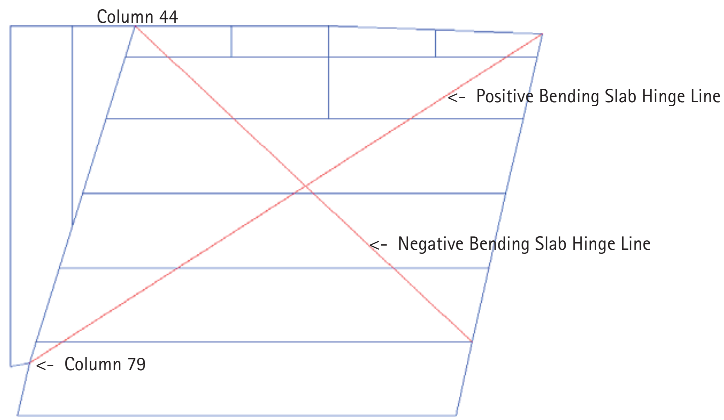


Figure B3.12 Projected plan of idealized floor deformation yield lines

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.

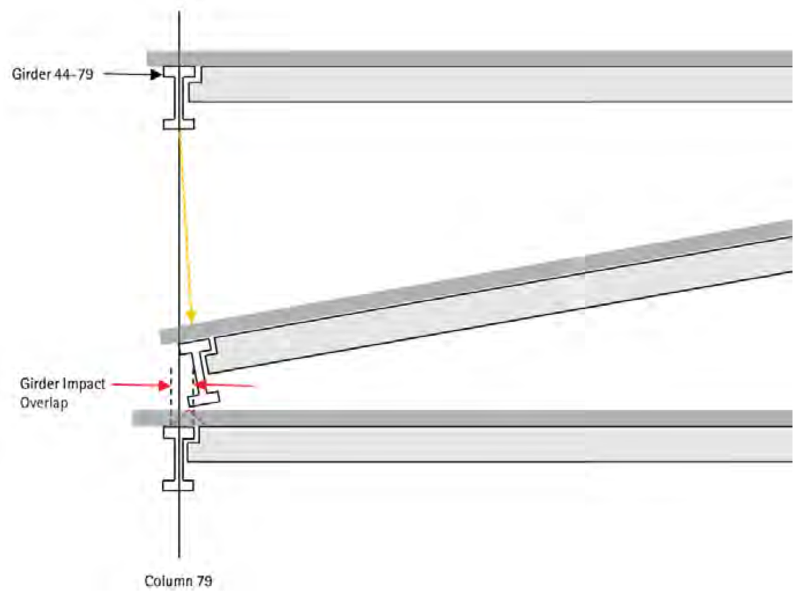


Figure B3.13 Girder impact overlap section diagram (north view)

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.

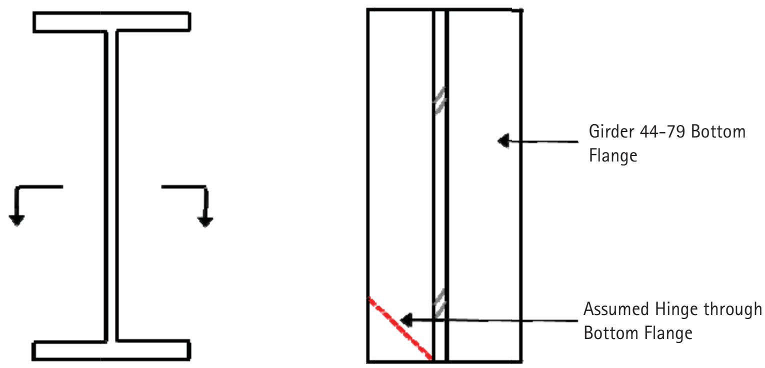


Figure B3.14 Assumed girder plastic hinging at impact (section and plan)

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.

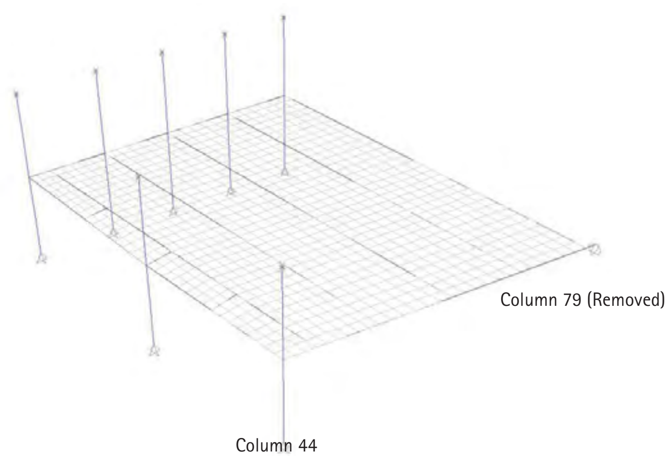


Figure B4.1 Partial floor SAP2000 potential energy model (northwest view)

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.

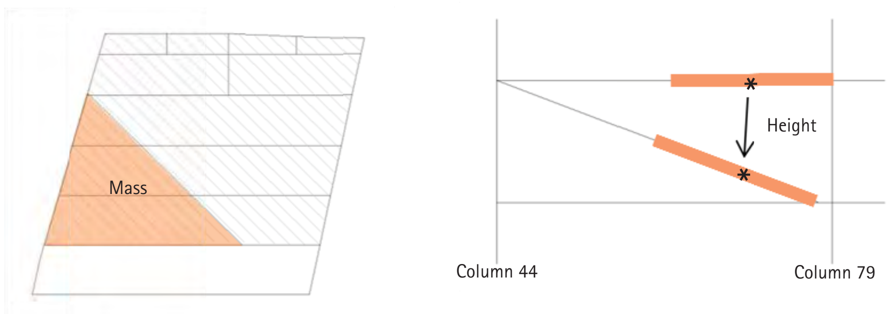


Figure B4.2 Tributary equivalent floor system area along hinge lines

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.

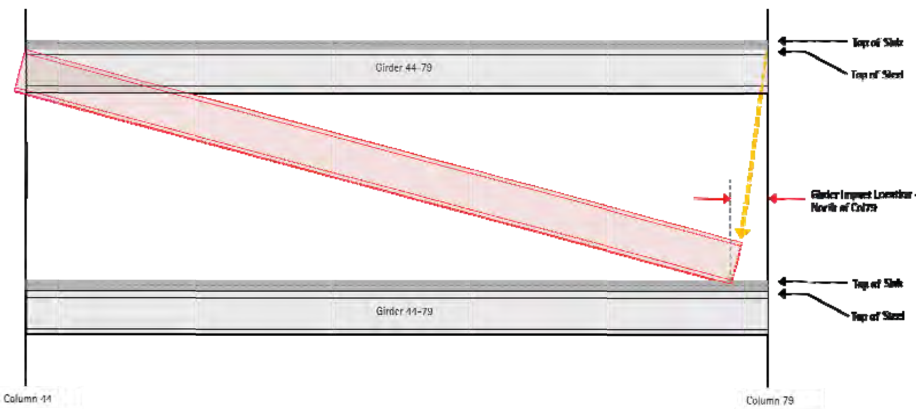


Figure B5.1 Girder 44-79 Simplified Rotation Diagram

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.

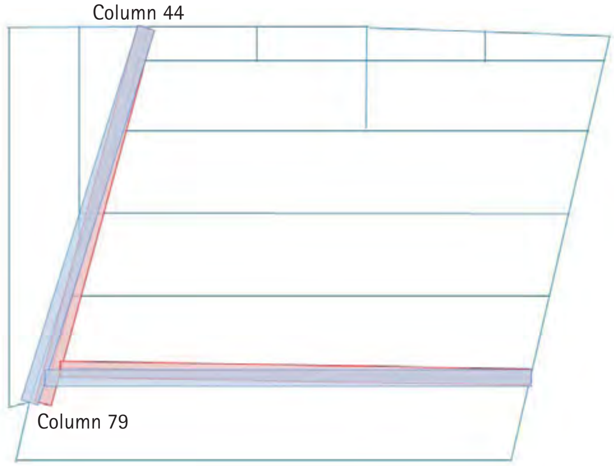


Figure B5.2 Girder 44-79 Impact Overlap Plan Diagram (position of intact girder below show in blue; position of rotated girder at impact shown in red)

Table B5.1 Girder impact overlap geometry

Floor	Flr-to-Flr 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

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

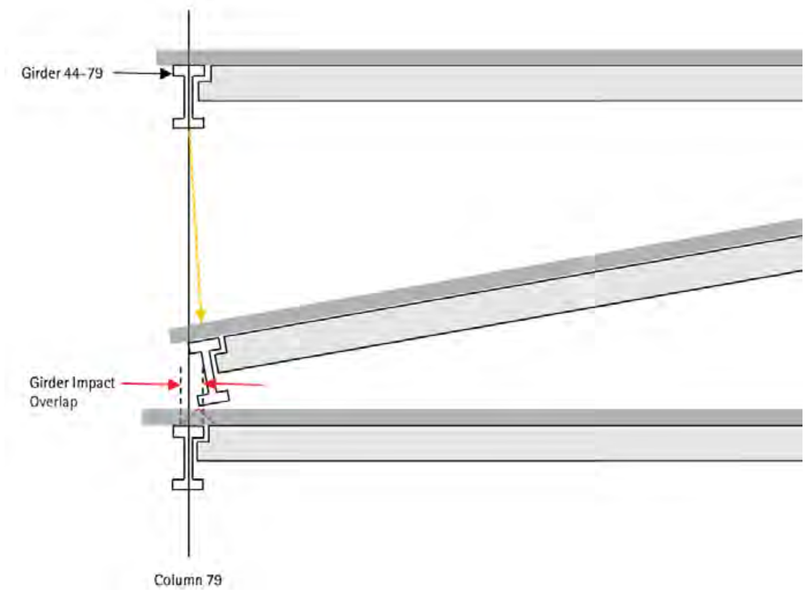


Figure B5.3 Girder impact overlap section diagram

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:

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

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

* Assumed; available shop drawings depict Column 79 prior to Floor 2 addition in NE corner

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

θ = angle from longitudinal axis of weld

Fexx= 77ksi expected for E70 electrodes

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.

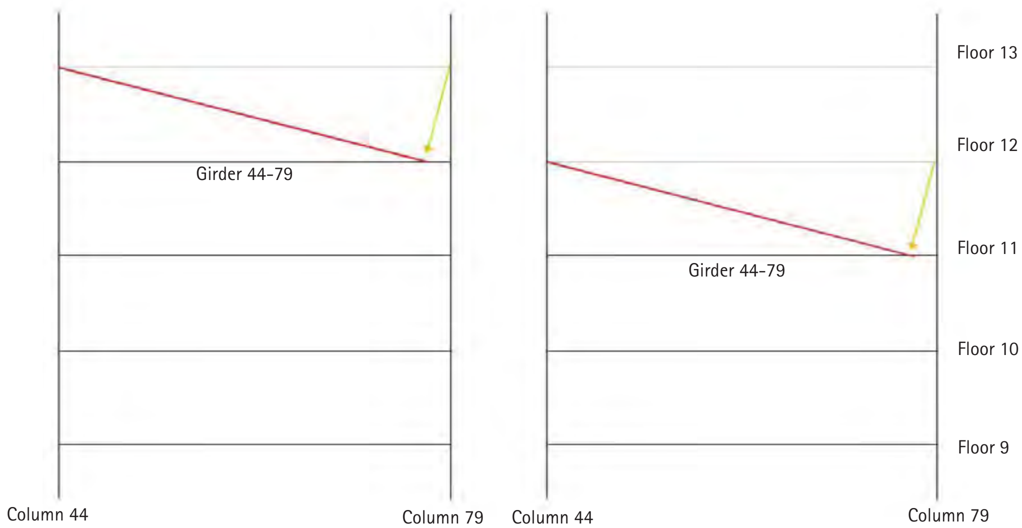


Figure B6.1 Vertical collapse propagation diagram

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

INITIAL ENERGY		ENERGY LOSS								IMPACT ENERGY	IMPACT FORCE AND FAILURE					
Level	Floor Level Potential Energy (k-in) i	South Bay Tensile Fracture Energy (k-in) ii	West Bay Tensile Fracture Energy (k-in) iii	Slab Boundary Rotations (k-in) iv	Slab Deformation (k-in) v	Slab Shear (k-in) vi	Girder Impact Corner Yielding (k-in) vii	Total Dissipated Energy (k-in) viii	Remaining Energy at Impact (k-in) ix	Girder 44-79 Stiffness Below at Impact (k/in) x	Equiv Static Force at Impact (kips) xi	Shear at Face of Col 79 (kips) xii		Shear Capacity of Col 79 Conn Below (kips) xiii	Connection Shear Failure? xiv
13	3818	17	14	147	70	14	82	345	3473	139	984	787	vs	632	
12	3818	17	14	147	70	14	82	345	3473	7627	7279	6915	vs	632	FAILURE
11	3818	17	14	147	70	14	82	345	3473	7627	7279	6915	vs	632	FAILURE
10	3818	17	14	147	70	14	82	345	3473	7627	7279	6915	vs	632	FAILURE
9	3818	17	14	147	70	14	82	345	3473	7002	6975	6626	vs	632	FAILURE
8	3879	17	14	147	70	14	82	345	3535	8112	7573	7194	vs	632	FAILURE
7	5169	31	25	518	302	23	60	959	4210	3782	5643	5361	vs	979	FAILURE
6	4360	-	15	160	105	15	53	348	4012	12708	10098	9593	vs	734	FAILURE
5	8406	56	47	2238	1071	33	494	3939	4467	11407	10095	9591	vs	1190	FAILURE
4	8475	17	15	244	105	15	91	488	7987	11759	13706	13020	vs	895	FAILURE
3	7626	17	-	168	111	9	105	410	7217	19044	16579	15750	vs	638	FAILURE
2	-	-	-	-	-	-		-	-	-	-	-		-	FAILURE

Notes

i Calculated based on floor slab gravity reaction at Col79 and reduced floor-to-floor height of equivalent failed slab section centroid

ii 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

iii See note ii above

iv Calculated using area under slab section plastic moment-rotation curves and assuming full plastic hinge formation along entire length of all boundaries

v Calculated using area under slab section plastic moment-rotation curves and assuming full plastic hinge formation along entire length of all idealized slab hinge lines

vi Concrete fracture mode assumed to be pure shear across entire slab edge length

vii Calculated assuming formation of full plastic hinge along corner of Girder 44-79 upon impact with floor below

viii Sum of South Bay Tensile Fracture, West Bay Tensile Fracture, Slab Boundary Rotations, Slab Deformation, and Slab Shear

ix Remaining Energy at Impact = Floor Level Potential Energy - Total Dissipated Energy

x Girder stiffness at level below failed slab calculated based on assumption of simply supported beam using K = 3*E*I*L / (I*a^2*b^2) where E = 29000ksi

xi Force = K * Deflection(D) where D = SQRT(2*Energy/K) based on PE= 0.5*K*D^2

xii By geometry of impact location

xiii Determined through series of AISC-prescribed calculations using expected material strengths

xiv If shear at face of column > shear capacity of connection below, shear failure assumed to occur

JA-4103

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.

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.

Table B8.1 Moment Rotation Curve Summary

FLOOR SLAB COLLAPSE ANALYSIS

MOMENT-ROTATION CURVE SUMMARY

					Point 1		Point2		Point 3	
				Label	Moment (k-in)	Rotation (1/in)	Moment (k-in)	Rotation (1/in)	Moment (k-in)	Rotation (1/in)
Floors 8-13	Slab Deformation	Positive	Typical Major	DP8	1.31	0.00004	6.22	0.00024	6.22	0.01263
		Negative	Angled Major	DN8	1.90	0.00006	1.36	0.00066	-	-
	North Boundary	Negative	Spandrel Major	N8	1.90	0.00006	6.17	0.00069	8.81	0.00760
	East Boundary	Negative	Spandrel Minor	E8	0.15	0.00003	6.47	0.00061	9.48	0.01140
	South Boundary	Positive	Typical Major	SP8	-	-	-	-	-	-
		Negative	Typical Major	SN8	-	-	-	-	-	-
	West Boundary	Positive	Typical Major	WP8	1.31	0.00004	6.22	0.00024	6.22	0.01502
		Negative	Typical Major	WN8	1.90	0.00006	0.96	0.00065	-	-
Floor 7	Slab Deformation	Positive	Typical Major	DP7	2.99	0.00003	16.80	0.00017	13.52	0.01741
		Negative	Angled Major	DN7	4.05	0.00004	17.64	0.00048	18.08	0.00569
	North Boundary	Negative	Spandrel Major	N7	4.05	0.00004	20.12	0.00051	27.94	0.00252
	East Boundary	Negative	Spandrel Minor	E7	1.98	0.00005	12.05	0.00082	16.96	0.00339
	South Boundary	Positive	Typical Major	SP7	2.99	0.00003	13.52	0.00017	16.80	0.01757
		Negative	Typical Major	SN7	4.05	0.00004	12.94	0.00044	13.50	0.00874
	West Boundary	Positive	Typical Major	WP7	2.99	0.00003	13.52	0.00017	16.80	0.01757
		Negative	Typical Major	WN7	4.05	0.00004	12.94	0.00044	13.50	0.00874
Floor 6	Slab Deformation	Positive	Typical Major	DP6	1.58	0.00004	7.05	0.00022	7.05	0.01675
		Negative	Angled Major	DN6	2.26	0.00005	1.51	0.00060	-	-
	North Boundary	Negative	Spandrel Major	N6	2.26	0.00005	7.18	0.00054	10.55	0.01277
	East Boundary	Negative	Spandrel Minor	E6	0.71	0.00009	2.91	0.00147	4.14	0.00443
	South Boundary	Positive	Typical Major	SP6	-	-	-	-	-	-
		Negative	Typical Major	SN6	-	-	-	-	-	-
	West Boundary	Positive	Typical Major	WP6	1.58	0.00004	7.05	0.00022	7.05	0.01676
		Negative	Typical Major	WN6	2.26	0.00005	1.06	0.00059	-	-
Floor 5	Slab Deformation	Positive	Typical Major	DP5	10.81	0.00002	29.92	0.00009	26.64	0.03322
		Negative	Angled Major	DN5	13.10	0.00002	62.03	0.00027	63.39	0.00525
	North Boundary	Negative	Spandrel Major	N5	13.10	0.00002	45.55	0.00024	65.23	0.00520
	East Boundary	Negative	Spandrel Minor	E5	9.57	0.00002	36.37	0.00029	53.00	0.00905
	South Boundary	Positive	Typical Major	SP5	10.81	0.00002	26.64	0.00009	29.92	0.03340
		Negative	Typical Major	SN5	13.10	0.00002	45.55	0.00024	47.37	0.00817
	West Boundary	Positive	Typical Major	WP5	10.81	0.00002	26.64	0.00009	29.92	0.03340
		Negative	Typical Major	WN5	13.10	0.00002	45.55	0.00024	47.37	0.00817
Floor 4	Slab Deformation	Positive	Typical Major	DP4	1.58	0.00004	7.05	0.00022	7.05	0.01675
		Negative	Angled Major	DN4	2.26	0.00005	1.51	0.00060	-	-
	North Boundary	Negative	Spandrel Major	N4	2.26	0.00005	7.18	0.00054	10.55	0.01277
	East Boundary	Negative	Spandrel Minor	E4	0.71	0.00009	2.91	0.00147	4.14	0.00443
	South Boundary	Positive	Typical Major	SP4	1.58	0.00004	7.05	0.00022	7.05	0.01676
		Negative	Typical Major	SN4	2.26	0.00005	1.06	0.00059	-	-
	West Boundary	Positive	Typical Major	WP4	1.58	0.00004	7.05	0.00022	7.05	0.01676
		Negative	Typical Major	WN4	2.26	0.00005	1.06	0.00059	-	-
Floor 3	Slab Deformation	Positive	Typical Major	DP3	1.58	0.00004	7.05	0.00022	7.05	0.01675
		Negative	Angled Major	DN3	2.26	0.00005	1.51	0.00060	-	-
	North Boundary	Negative	Spandrel Major	N3	2.26	0.00005	7.18	0.00054	10.55	0.01277
	East Boundary	Negative	Spandrel Minor	E3	0.71	0.00009	2.91	0.00147	4.14	0.00443
	South Boundary	Positive	Typical Major	SP3	1.58	0.00004	7.05	0.00022	7.05	0.01676
		Negative	Typical Major	SN3	2.26	0.00005	1.06	0.00059	-	-
	West Boundary	Positive	Typical Major	WP3	-	-	-	-	-	-
		Negative	Typical Major	WN3	-	-	-	-	-	-
Floor 2	Slab Deformation	Positive	Typical Major	DP2	1.58	0.00004	7.05	0.00022	7.05	0.01675
		Negative	Angled Major	DN2	2.26	0.00005	1.51	0.00060	-	-
	North Boundary	Negative	Spandrel Major	N2	2.26	0.00005	7.18	0.00054	10.55	0.01277
	East Boundary	Negative	Spandrel Minor	E2	0.71	0.00009	2.91	0.00147	4.14	0.00443
	South Boundary	Positive	Typical Major	SP2	1.58	0.00004	7.05	0.00022	7.05	0.01676
		Negative	Typical Major	SN2	2.26	0.00005	1.06	0.00059	-	-
	West Boundary	Positive	Typical Major	WP2	-	-	-	-	-	-
		Negative	Typical Major	WN2	-	-	-	-	-	-

Table B8.2 Plastic Hinge Energy Calculations

FLOOR SLAB COLLAPSE ANALYSIS

PLASTIC HINGE ENERGIES

		Bending Direction	Hinge Length (in)	Energy/ft Hinge (k-in)	Total Energy (k-in)		
Floors 8-13	Slab Deformation	Positive Bending	888	0.9	69.1	Sum - Deformation (k-in)	
		Negative Bending	700	0.0	0.7	70	
	North Slab Boundary	Negative Bending	785.2	0.7	42.7		
	East Slab Boundary	Negative Bending	608.7	1.1	53.6		
	South Slab Boundary	Positive Bending	-	-	-		
		Negative Bending	-	-	-		
	West Slab Boundary	Positive Bending	547	1.1	50.7	Sum - Boundary (k-in)	Total - Levels 8-13 (k-in)
		Negative Bending	530	0.0	0.5	147	217
Floor 7	Slab Deformation	Positive Bending	888	3.2	233.3	Sum - Deformation	
		Negative Bending	700	1.2	68.6	302	
	North Slab Boundary	Negative Bending	786.8	0.6	42.6		
	East Slab Boundary	Negative Bending	608.7	0.5	26.0		
	South Slab Boundary	Positive Bending	647.8	3.2	171.6		
		Negative Bending	647.8	1.4	73.3		
	West Slab Boundary	Positive Bending	548	3.2	145.2	Sum - Boundary (k-in)	Total - Level 7 (k-in)
		Negative Bending	525	1.4	59.4	518	820
Floor 6	Slab Deformation	Positive Bending	889	1.4	104.3	Sum - Deformation (k-in)	
		Negative Bending	713	0.0	0.8	105	
	North Slab Boundary	Negative Bending	791.2	1.3	87.6		
	East Slab Boundary	Negative Bending	608.8	0.2	7.9		
	South Slab Boundary	Positive Bending	-	-	-		
		Negative Bending	-	-	-		
	West Slab Boundary	Positive Bending	543	1.4	63.8	Sum - Boundary (k-in)	Total - Level 6 (k-in)
		Negative Bending	539	0.0	0.5	160	265
					0.3		
Floor 5	Slab Deformation	Positive Bending	891.0	11.3	836.2	Sum - Deformation (k-in)	
		Negative Bending	730.0	3.9	234.8	1071	
	North Slab Boundary	Negative Bending	869.0	3.4	244.4		
	East Slab Boundary	Negative Bending	609.0	4.8	242.3		
	South Slab Boundary	Positive Bending	647.0	14.8	798.5		
		Negative Bending	648.0	4.5	243.0		
	West Slab Boundary	Positive Bending	538.0	11.3	507.6	Sum - Boundary (k-in)	Total - Level 5 (k-in)
		Negative Bending	539.0	4.5	202.2	2238	3309
Floor 4	Slab Deformation	Positive Bending	891.0	1.4	104.6	Sum - Deformation (k-in)	
		Negative Bending	730.0	0.0	0.8	105	
	North Slab Boundary	Negative Bending	870.0	1.3	96.4		
	East Slab Boundary	Negative Bending	609.0	0.2	7.9		
	South Slab Boundary	Positive Bending	647.0	1.4	76.0		
		Negative Bending	648.0	0.0	0.6		
	West Slab Boundary	Positive Bending	537.0	1.4	63.1	Sum - Boundary (k-in)	Total - Level 4 (k-in)
		Negative Bending	539.0	0.0	0.5	244	350
Floor 3	Slab Deformation	Positive Bending	935.0	1.4	109.7	Sum - Deformation (k-in)	
		Negative Bending	770.0	0.0	0.8	111	
	North Slab Boundary	Negative Bending	696.0	1.3	77.1		
	East Slab Boundary	Negative Bending	609.0	0.2	7.9		
	South Slab Boundary	Positive Bending	700.0	1.4	82.2		
		Negative Bending	700.0	0.0	0.7		
	West Slab Boundary	Positive Bending	-	-	-	Sum - Boundary (k-in)	Total - Level 3 (k-in)
		Negative Bending	-	-	-	168	278
Floor 2	Slab Deformation	Positive Bending	913.0	1.4	107.1	Sum - Deformation (k-in)	
		Negative Bending	829.0	0.0	0.9	108	
	North Slab Boundary	Negative Bending	692.0	1.3	76.7		
	East Slab Boundary	Negative Bending	609.0	0.2	7.9		
	South Slab Boundary	Positive Bending	681.0	1.4	80.0		
		Negative Bending	681.0	0.0	0.6		
	West Slab Boundary	Positive Bending	-	-	-	Sum - Boundary (k-in)	Total - Level 2 (k-in)
		Negative Bending	-	-	-	165	273

Guy Nordenson and Associates

APPENDIX C

Interior Column Stability Analysis Report

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

By
Guy Nordenson PE, SE

Structural Engineers LLP

225 Varick Street 6th Flr
New York NY 10014 USA
Tel 212 766 9119
Fax 212 766 9016
www.nordenson.com

TABLE OF CONTENTS

		Page
C1.0	**INTRODUCTION**	**C1**
C2.0	**BACKGROUND ON THE ASSESSMENT OF COLUMN STABILITY**	**C2**
C2.1	Stiffness and Strength Requirements for Adequate Lateral Bracing	C2
C2.2	Actual Conditions of WTC7 Columns	C4
C2.3	Computer Analysis Methods to Assess Stability	C6
C3.0	**INTERIOR COLUMN STABILITY ANALYSES**	**C8**
C3.1	Basis of Buckling Sequence	C8
C3.2	Interior Column Stability Analysis Input	C8
C3.3	Interior Column Stability Analysis Results and Interpretation	C13
C4.0	**ADDITIONAL STABILITY STUDIES**	**C15**
C4.1	Evaluation of Column Stability with 2% Bracing Provided	C15
C5.0	**INTERIOR COLUMN CONNECTION TYPE CATALOGUE**	**C16**
C6.0	**INTERIOR COLUMN BRACING CAPACITY TABLES**	**C41**

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_{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 ($4*P_{cr}$). 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_{cr} (Figure C2.1a-d) (Ref 15).

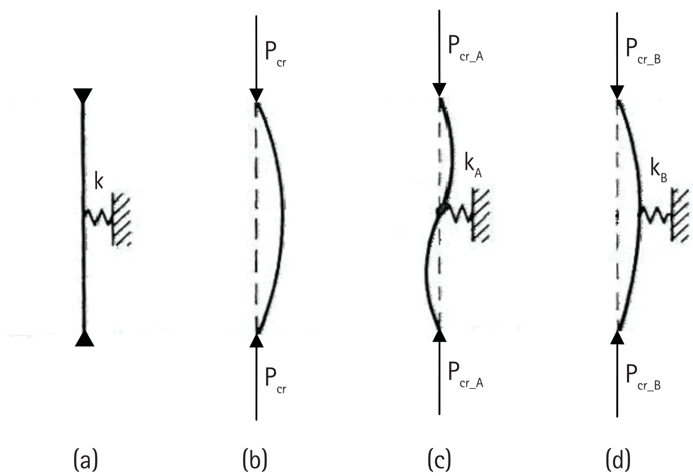


Figure C2.1a-d Effect of brace stiffness on the buckling of an ideal column. (a) unloaded column with lateral brace (b) buckled column without intermediate brace (c) buckled shape for stiff spring as intermediate support (d) buckled shape for flexible spring as intermediate support. $P_{cr\_A} > P_{cr\_B}$ (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).

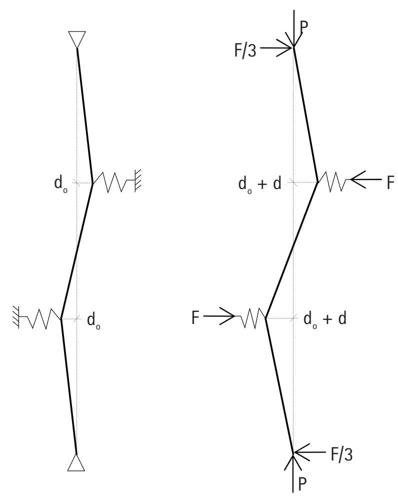


Figure C2.2 Lateral forces and displacements resulting from initial crookedness of column

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.

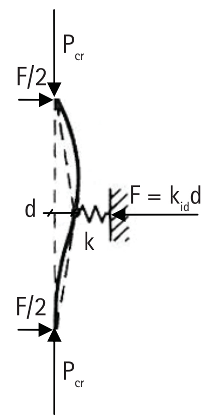


Figure C2.3 Calculation of required stiffness (k_{id}=2P_{cr}/l) for simple ideal case using small deflection theory (Ref 15)

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).

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 process[1]. 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.

---

[1] 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 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).

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.

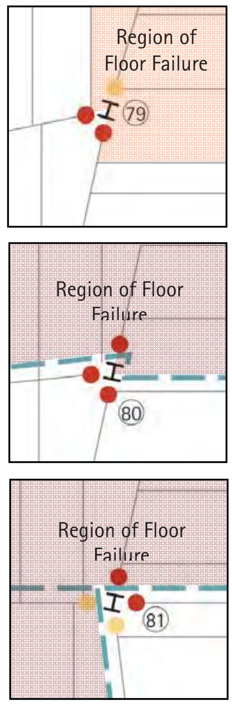


Figure C3.1 Sketches of interior column bracing conditions as floor collapse progresses (collapsed slabs are indicated in orange and pink)

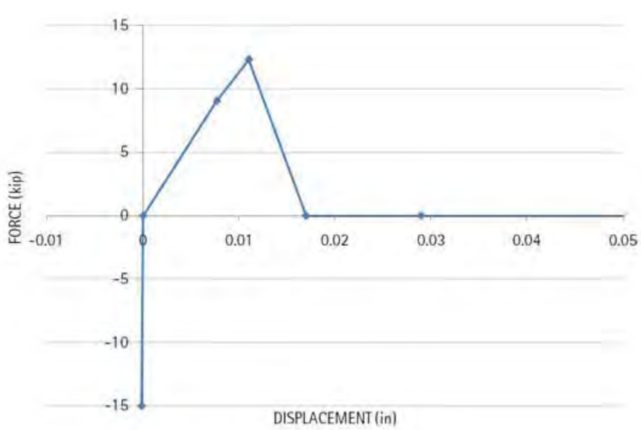


Figure C3.2 Axial force-displacement curve for a link corresponding to a 14.5"-long double-angle knife connection

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

C5.1 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 58

COLUMN 58

		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x99	W30x135	Knife	L4x3x3/8	20.5	1.41	W21x50	Knife	L4x3x3/8	11.5	0.79	W30x108	Header	L3x3x5/16	14.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 47	W14x109	-	-	-	-	-	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x5/16	17.5	-	W33x201	Header Pl	Pl 7x1/2	27	-
Floor 46		W36x260	Header	L4x4x1/2	27.5	1.90	W21x44	Knife	L4x3x3/8	11.5	0.79	W30x108	Header	L3x3x5/16	20.5	-	W24x55	Header Pl	Pl 7.5x3/8	12	-
Floor 45	W14x145	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 44		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 43	W14x193	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 42		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5		W16x26	Header	L3x3x5/16	8.5	-
Floor 41	W14x233	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 40		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 39	W14x283	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 38		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 37	W14x311	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 36		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 35	W14x370	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 34		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 33	W14x426	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 32		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 31	W14x455	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	--
Floor 30		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 29	W14x500	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 28		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 27	W14x500	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 26		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 25	W14x550	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 24		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 23	W14x550	W33x118	Knife	L4x3x3/8	17.5	1.21	W21x50	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 22		W33x118	Knife	L4x3x3/8	17.5	1.21	W21x50	Knife	L4x3x3/8	11.5	0.79	W24x68	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 21	W14x605	W33x118	Knife	L4x3x3/8	17.5	1.21	W21x50	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 20		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 19	W14x605	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 18		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 17	W14x665	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 16		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 15	W14x665	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 14		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 13	W14x730	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 12		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 11	W14x730	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 10		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 9	W14x730	W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W27x161	Header	L3x3x3/8	17.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 8		W30x116	Knife	L4x3x3/8	17.5	1.21	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W16x26	Header	L3x3x5/16	8.5	-
Floor 7	Sits on Transfer	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.2 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 59

COLUMN 59																						
		North					South					East					West					
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W21x50	Knife	L4x3x3/8	11.5	0.79	W18x35	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L4x3x5/16 bent	14.5	-	-	-	-	-	-
Floor 47	W14x90	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 46		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 45	W14x99	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 44		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 43	W14x109	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	-	-	-	-	-
Floor 42		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 41	W14x132	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	-	-	-	-	-
Floor 40		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 39	W14x145	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	-	-	-	-	-
Floor 38		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 37	W14x159	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 36		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 35	W14x193	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 34		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 33	W14x211	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 32		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 31	W14x233	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 30		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 29	W14x257	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 28		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 27	W14x283	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 26		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 25	W14x311	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 24		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 23	W14x311	W21x50	Knife	L4x3x3/8	11.5	0.79	W18x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 22		W21x50	Knife	L4x3x3/8	11.5	0.79	W18x35	Knife	L4x3x3/8	8.5	0.59	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 21	W14x342	W21x50	Knife	L4x3x3/8	11.5	0.79	W18x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 20		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 19	W14x370	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 18		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 17	W14x398	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 16		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 15	W14x426	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 14		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 13	W14x426	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 12		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 11	W14x455	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 10		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 9	W14x500	W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 8		W21x44	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 7	Sits on Transfer	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.3 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 60

COLUMN 60

		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W18x35	Knife	L4x3x3/8	11.5	0.79	W18x35	Knife	L4x3x3/8	8.5	0.59	W24x55	Header	L4x3x5/16 bent	11.5	-	W14x22	Header	L4x3x3/8	8.5	-
Floor 47	W14x90	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3.5x5/16 bent L5x3.5x5/16 bent	11.5	-	-	-	-	-	-
Floor 46		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x94	Header	L4x4x5/16 bent	17.5	-	-	-	-	-	-
Floor 45	W14x99	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3.5x5/16 bent-L5x3.5x5/16 bent	11.5	-	-	-	-	-	-
Floor 44		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3.5x5/16 bent-L5x3.5x5/16 bent	11.5	-	-	-	-	-	-
Floor 43	W14x109	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 42		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 41	W14x132	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 40		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 39	W14x159	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 38		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 37	W14x193	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 36		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 35	W14x233	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 34		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 33	W14x233	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 32		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 31	W14x257	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 30		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 29	W14x283	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 28		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 27	W14x311	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 26		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x77	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 25	W14x342	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 24		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 23	W14x370	W18x35	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 22		W18x35	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 21	W14x398	W18x35	Knife	L4x3x3/8	8.5	0.59	W16x77	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 20		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 19	W14x426	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 18		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 17	W14x455	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 16		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 15	W14x500	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 14		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 13	W14x500	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 12		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 11	W14x550	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 10		W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 9	W14x550	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 8		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x77	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-	-	-	-	-	-
Floor 7	W14x550	W18x35	Knife	L4x3x3/8	8.5	0.59	W24x94	Knife	L4x3x3/8	17.5	1.21	W24x68	Header	L4x3.5x5/16 bent-L5x3.5x5/16 bent	11.5	-	-	-	-	-	-
Floor 6		W18x40	Knife	L4x3x3/8	11.5	0.79	W18x40	Knife	L4x3x3/8	11.5	0.79	W18x35	Header	L4x4x3/8 bent	8.5	-	W14x30	Header	L5x3x5/16	8.5	-
Floor 5	W14x605	W21x44	Knife	L4x3x3/8	14.5	1.00	W30x173	Knife	L4x4x3/8	23.5	1.62	W30x173	Header	L4x4x1/2	24.5	-	-	-	-	-	-
Floor 4		W21x44	Knife	L4x3x3/8	11.5	0.79	W30x99	Knife	L4x3x3/8	14.5	1.00	W12x19	Header	No Information Available	-	--	-	-	-	-	-
Floor 3	W14x665	W24x62	Knife	L4x3x3/8	21	1.45	W36x94	Knife	L4x3x3/8	27	1.86	W33x118	Header	9.25x3/8 bent	24.5	-	-	-	-	-	-
Floor 2		W24x55	Knife	L4x3x3/8	12	0.83	W33x130	Knife	L4x3x3/8	18	1.24	W21x101	Header	L3x3x5/16	17.5	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.4 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 61

COLUMN 61		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Bulkhead	W14x90	-	-	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.59	W16x26	Header	L3x3x5/16	11.5	-	W12x19	Header	L3x3x5/16	8.5	-
Roof		W24x68	Knife	L4x3x3/8	11.5	0.79	W18x40	Knife	L4x3x3/8	11.5	0.79	W30x99	Header	No Information Available	-	-	W30x108	Header	L3x3x5/16	14.5	-
Floor 47	W14x99	W12x16	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W27x84	Header	L4x3x3/8	21	-	W27x161	Header	L4x3x5/16	18	-
Floor 46		W36x150	Knife	L4x3x3/8	17.5	1.21	W18x40	Knife	L4x3x3/8	11.5	0.79	W33x130	Header	L4x3x1/2	24	-	W30x108	Header	L4x3x3/8	18	-
Floor 45	W14x132	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 44		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 43	W14x159	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 42		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	
Floor 41	W14x193	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 40		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 39	W14x211	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 38		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 37	W14x233	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 36		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 35	W14x283	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 34		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 33	W14x342	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 32		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 31	W14x370	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 30		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 29	W14x398	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 28		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 27	W14x426	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 26		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 25	W14x455	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 24		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 23	W14x500	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 22		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 21	W14x550	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x55	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 20		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 19	W14x605	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 18		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 17	W14x605	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 16		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 15	W14x665	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 14		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 13	W14x665	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 12		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 11	W14x730	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 10		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 9	W14x730	W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W27x161	Header	L3x3x3/8	17.5	-
Floor 8		W24x55	Seated	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 7	W14x500	MG-27	Knife	L7x4x5/8	39.5	2.72	W14x342	Truss Cnx	-	-	-	W36x135	Header	L3x3x5/16	23.5	-	W27x161	Header	L3x3x5/16	18.5	-
Floor 6		W21x44	Knife	L4x3x3/8	11.5	0.79	W14x342	Truss Cnx	-	-	-	W21x50	Header	L4x3x5/16	18	-	W24x76	Header	L4x4x3/8	14.5	-
Floor 5		W24x68	Knife	L4x3x3/8	11.5	0.79	W14x342	Truss Cnx	-	-	-	W24x84	Header	L4x3x5/16	18.5	-	W33x118	Header	L4x4x3/8	23.5	-
Floor 4	W14x93	W27x84	Knife	L4x3x3/8	14.5	1.00	W12x19	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L3x3x5/16	20.5	-	W33x118	Header	L3x3x5/16	23.5	-
Floor 3	Sits on existing Col	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.5 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 62

COLUMN 62

		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x62	Header	L5x3.5x3/8	40.5	-	W33x130	Header	L4x3x3/8	26.5	1.83	W30x99	Header	L4x3x5/16	14.5	-
Floor 47	W14x90	W18x40	Knife	L4x3x3/8	14.5	1.00	W12x90	Knife	L4x3x3/8	8.5	0.59	W24x76	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 46		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L3x3x5/16	8.5	-
Floor 45	W14x99	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 44		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 43	W14x109	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	No Information Available	-	-
Floor 42		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 41	W14x132	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	No Information Available	-	-
Floor 40		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 39	W14x143	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	No Information Available	-	-
Floor 38		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 37	W14x159	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 36		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 35	W14x176	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 34		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 33	W14x193	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 32		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 31	W14x211	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 30		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 29	W14x233	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 28		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 27	W14x257	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 26		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 25	W14x283	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 24		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 23	W14x311	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 22		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x44	Knife	L4x3x3/8	11.5	0.79	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 21	W14x342	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x44	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 20		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 19	W14x370	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 18		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 17	W14x398	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 16		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 15	W14x426	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 14		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 13	W14x426	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 12		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 11	W14x455	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 10		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 9	W14x455	W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W24x76	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 8		W18x40	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x40	Knife	L4x3x3/8	11.5	0.79	W21x44	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	-
Floor 7	TRUSS #3	W14x342	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W21x50	Knife (offset)	L4x3x3/8	16.5	1.14	W24x68	Header	L3x3x5/16	11.5	-
Floor 6		W14x342	Truss Con	-	-	-	W24x55	Knife	L4x3x3/8	14.5	1	W21x50	Knife (offset)	L4x3x3/8	11.5	0.79	W24x76	Header	L3x3x5/16	14.5	-
Floor 5	W14x233	W14x342	Truss Con	-	-	-	W18x35	Knife	L4x3x3/8	8.5	0.59	W21x50	Knife (offset)	L4x3x3/8	11.5	0.79	W30x173	Header	L4x4x1/2	24.5	-
Floor 4		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W24x62	Knife (offset)	L4x3x3/8	11.5	0.79	W12x19	Header	L3x3x5/16	5.5	-
Floor 3	W14x311	W18x76	Knife	L4x3x1/2-7/16 weld	15	1.45	W14x22	Knife	L4x3x3/8	8.5	0.59	W24x55	Fin (offset)	Pl 7x3/4	19	-	W12x14	Header	L3x3x5/16	8.5	-
Floor 2		W18x35	Knife	L4x3x3/8	12	0.83	W16x71	Knife	L4x3x3/8	15	1.03	W21x62	Fin (offset)	Pl 6.5x3/4	18	-	W18x76	Header	L3x3x5/16	11.5	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.6 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 63

COLUMN 63		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W24x62	Knife	L4x3x3/8	11.5	0.79	W14x22	Header	L4x3x5/16 bent-L5x3x5/16 bent	8.5	–	W21x44	Header	L4x3x5/16	11.5	–	W24x55	Header	L3x3x5/16	11.5	–
Floor 47	W14x90	W18x40	M-Conn	–	–	–	W14x22	Header	L4x3x5/16 bent	9.5	–	W24x76	Header	L4x3x5/16	17.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 46		W12x19	Knife	L4x3x3/8	8.5	0.59	W36x150	Header	L4x3x5/16	17.5	–	W24x62	Header	L4x3x5/16 bent	12	–	W27x94	Header	L4x3x1/2 bent-L5x3x1/2 bent	21.5	–
Floor 45	W14x99	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16 bent	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 44		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16 bent	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 43	W14x109	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 42		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 41	W14x132	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 40		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 39	W14x159	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 38		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 37	W14x176	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 36		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 35	W14x193	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 34		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 33	W14x211	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 32		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 31	W14x233	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 30		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 29	W14x257	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 28		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 27	W14x283	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 26		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 25	W14x311	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 24		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 23	W14x342	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W21x44	Header	L4x3x5/16	11.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 22		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W21x44	Header	L4x3x5/16	11.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 21	W14x370	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W21x44	Header	L4x3x5/16	11.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 20		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 19	W14x398	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 18		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 17	W14x426	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 16		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 15	W14x426	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 14		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 13	W14x455	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 12		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 11	W14x455	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 10		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 9	W14x500	W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 8		W12x19	Knife	L4x3x3/8	8.5	0.59	W16x31	Seated	–	–	–	W18x40	Header	L4x4x5/16	8.5	–	W21x50	Header	L4x3x5/16 bent-L5x3x5/16 bent	11.5	–
Floor 7	W14x500	W16x26	Knife	L4x3x3/8	8.5	0.59	W24x55	Knife	L4x3x3/8	11.5	0.79	W18x65	Header	L4x3x5/16	11.5	–	W24x68	Header	L4x3.5x5/16 L5x3.5x5/16	11.5	–
Floor 6		W24x55	Fin	1Pl 5x1/2	18.5	–	W24x55	Fin	1Pl 5x1/2	18.5	–	W18x35	Header	L3x3x5/16	8.5	–	W18x35	Header	L4x4x3/8	8.5	–
Floor 5	W14x550	W18x35	Knife	L4x3x3/8	8.5	0.59	W27x84	Knife	L4x3x3/8	14.5	1.00	W24x131	Header	L4x4x3/8	19	–	W30x173	Header	L4x4x1/2	24.5	–
Floor 4		W16x31	Knife	L4x3x3/8	8.5	0.59	–	–	–	–	–	W21x50	Header	L3x3x5/16	11.5	–	W12x19	Header	L3x3x5/16	8.5	–
Floor 3	W14x665	W14x22	Knife	L4x3x3/8	9	0.62	W24x55	Knife	L4x3x3/8	12	0.83	W24x76	Header	L4x3x5/16	14.5	–	W33x118	Header	9.25x3/8 bent	24.5	–
Floor 2		W16x71	Knife	L4x3x3/8	9	0.62	W16x67	Knife	L4x3x3/8	9	0.62	W18x40	Header	L3x3x5/16	8.5	–	–	–	–	–	–

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

JA-4132

C5.7 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 64

COLUMN 64		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Bulkhead	W14x90	-	-	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.59	W16x26	Header	L3x3x5/16	8.5	-	W12x19	Header	L3x3x5/16	11.5	-
Roof		-	-	-	-	-	W12x16	Knife	L4x3x3/8	14.5	1.00	W36x150	Header	L4x3x1/2	21.5	-	W30x99	Header	No Information Available	-	-
Floor 47	W14x99	-	-	-	-	-	W36x150	Knife	L4x4x3/8	29.5	2.03	W36x150	Header	L3x3x5/16	20.5	-	W27x84	Header	L4x3x3/8	21	-
Floor 46		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W30x173	Header	L4x3x1/2	21	-	W33x130	Header	L4x3x1/2	24	-
Floor 45	W14x120	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 44		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 43	W14x145	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 42		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 41	W14x176	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 40		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 39	W14x211	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 38		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 37	W14x257	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 36		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 35	W14x283	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 34		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 33	W14x311	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 32		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 31	W14x342	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 30		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	No Information Available	-	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 29	W14x370	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W18x97	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 28		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 27	W14x398	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 26		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 25	W14x426	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 24		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 23	W14x455	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 22		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 21	W14x500	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W18x106	Header	L3x3x5/16	11.5	-	W24x55	Header	L3x3x5/16	11.5	-
Floor 20		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 19	W14x550	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 18		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 17	W14x550	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 16		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 15	W14x605	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 14		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 13	W14x605	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 12		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 11	W14x665	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 10		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 9	W14x665	-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 8		-	-	-	-	-	W12x35	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 7	W14x730	W16x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Truss Cnx	-	-	-	W36x300	Header	Pl 9x5/8	24.5	-	W36x135	Header	Pl 8.5x3/4	39.5	-
Floor 6		-	-	-	-	-	W24x76	Fin	Pl 4.25x1/2	15.25	-	W36x210	Truss Cnx	-	-	-	W21x50	Header	L3x3x5/16	14.5	-
Floor 5	W14x730	-	-	-	-	-	W24x76	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-	W24x84	Header	L3x3x5/16	18.5	-
Floor 4		-	-	-	-	-	W24x76	Fin	Pl 4.25x1/2	15	-	W36x260	Truss Cnx	-	-	-	W27x94	Header	L3x3x5/16	20.5	-
Floor 3	Existing Col	-	-	-	-	-	W24x76	Fin	Pl 11x1	24.5	-	-	Truss Cnx	-	-	-	Existing	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.8 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 65

COLUMN 65		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W30x99	Knife	L4x3x3/8	14.5	1	W24x62	Knife	L4x3x3/8	17.5	1.21	W33x130	Header	L3x3x5/16	20.5	-	W33x130	Knife	L4x3x3/8	23.5	-
Floor 47	W14x90	W36x150	Knife	L4x4x3/8	29.5	2.03	W33x118	Knife	L4x4x3/8	26.5	1.83	W36x194	Header	L3x3x5/16	26.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 46		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 45	W14x90	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 44		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 43	W14x99	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 42		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 41	W14x99	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 40		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 39	W14x109	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 38		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 37	W14x109	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 36		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 35	W14x120	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	No Information Available	-	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 34		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	No Information Available	-	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 33	W14x132	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	No Information Available	-	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 32		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	No Information Available	-	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 31	W14x145	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 30		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 29	W14x145	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 28		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 27	W14x159	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 26		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 25	W14x159	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 24		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 23	W14x176	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 22		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 21	W14x176	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 20		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 19	W14x193	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 18		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 17	W14x193	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 16		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 15	W14x211	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 14		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 13	W14x211	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 12		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 11	W14x233	W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 10		W12x35	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 9	W14x233	W18x40	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 8		W18x40	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	11.5	-
Floor 7	W14x665	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	MG-28	Header	L3x3x5/16	8.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 6		W24x76	Fin	Pl 4.25x1/2	15.25	-	W24x76	Fin	Pl 4.25x1/2	21.25	-	W24x68	Header	L3x3x5/16	12.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 5	W14x730	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	W36x150	Header	L3x3x5/16	29.5	-	W21x50	Header	L3x3x5/16	11.5	-
Floor 4		W24x76	Fin	Pl 4.25x1/2	15.25	-	W24x76	Fin	Pl 4.25x1/2	21	-	W8x31	Header	L3x3x5/16	8.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 3	W14x730	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	MG-31	Header	L3x3x5/16	23.5	-	W24x55	Header	L3x3x5/16	18.5	-
Floor 2		W24x76	Fin	Pl 4.25x1/2	21	-	W24x76	Fin	Pl 7.25x3/4	18	-	W30x99	Header	L3x3x5/16	17.5	-	W21x62	Header	L3x3x5/16	11.5	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.9 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 66

COLUMN 66		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W24x62	Knife	L4x3x3/8	11.5	0.79	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 47	W14x90	W33x118	M-Conn	-	-	-	-	-	-	-	-	W27x94	Header	L3x3x5/16	17.5	-	W24x76	Header	L3x3x5/16	17.5	-
Floor 46		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W33x130	Header	L3x3x5/16	20.5	-	W24x62	Header	L3x3x5/16	20.5	-
Floor 45	W14x99	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 44		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 43	W14x109	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 42		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 41	W14x132	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 40		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 39	W14x145	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 38		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 37	W14x159	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 36		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 35	W14x176	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 34		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 33	W14x193	W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 32		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 31	W14x211	W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 30		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 29	W14x233	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 28		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 27	W14x257	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 26		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 25	W14x283	W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 24		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W18x71	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 23	W14x283	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 22		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 21	W14x311	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W21x44	Header	L3x3x5/16	11.5	-
Floor 20		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 19	W14x342	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 18		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 17	W14x370	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W18x71	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 16		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 15	W14x398	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 14		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	8.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 13	W14x398	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 12		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 11	W14x426	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 10		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 9	W14x426	W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 8		W12x19	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W21x68	Header	L3x3x5/16	11.5	-	W18x40	Header	L3x3x5/16	8.5	-
Floor 7	W14x730	W24x76	Truss Cnn	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.58	W36x210	Header	Pl 7x5/8	21	-	W18x65	Header	L3x3x5/16	11.5	-
Floor 6		W24x76	Fin	Pl 4.25x1/2	21	-	W21x44	Fin	Pl 4.75x1/2	18	-	W36x210	Truss Cnn	-	-	-	W18x35	Header	L3x3x5/16	8.5	-
Floor 5	W14x730 + 2Web Pl's 12.5x3	W24x76	Truss Cnn	-	-	-	-	-	-	-	-	W36x300	Truss Cnn	-	-	-	W24x131	Knife	L4x3x3/8	20	1.38
Floor 4		W24x76	Fin	Pl 4.5x1/2	21	-	-	-	-	-	-	W8x31	Truss Cnn	-	-	-	W21x50	Knife	L4x3x3/8	11.5	0.79
Floor 3	W14x730 + 2Web Pl's 12.5x3	W24x76	Truss Cnn	-	-	-	-	-	-	-	-	W36x300	Truss Cnn	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 2		W24x76	Fin	Pl 7.5x3/4	18	-	-	-	-	-	-	W36x300	Truss Cnn	-	-	-	W18x40	Knife	L4x3x3/8	11.5	0.79

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.10 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 67

COLUMN 67		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	-	-	-	-	-	W24x55	Knife	L4x3x3/8	8.5	0.59	W30x116	Header	L3x3x5/16	14.5	-	W36x150	Header	L3x3x5/16	29.5	-
Floor 47	W14x99	-	-	-	-	-	W36x150	Knife	L4x4x3/8	29.5	2.03	W24x176	Header	L3x3x5/16	14.5	-	W36x150	Header	L3x3x5/16	20.5	-
Floor 46		-	-	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.58	W30x173	Header	L4x3x1/2	24	-	W36x230	Header	L4x3x1/2	21.5	-
Floor 45	W14x132	-	-	-	-	-	W18x35	Knife	L4x3x3/8	14.5	1.00	W30x132	Header	L3x3x5/16	20.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 44		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 43	W14x159	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x146	Header	L3x3x5/16	17.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 42		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 41	W14x193	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W27x146	Header	L3x3x5/16	17.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 40		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 39	W14x233	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W27x146	Header	L3x3x5/16	17.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 38		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 37	W14x283	-	-	-	-	-	W36x280	Seated	-	-	-	W36x170	Header	L3x3x3/8	38.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 36		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 35	W14x311	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 34		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 33	W14x342	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	No Information Available	-	-
Floor 32		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	No Information Available	-	-
Floor 31	W14x370	-	-	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.58	W24x146	Header	L3x3x3/8	17.5	-	W24x76	Header	No Information Available	-	-
Floor 30		-	-	-	-	-	W33x118	Knife	L4x4x3/8	26.5	1.82	W27x94	Header	L3x3x5/16	14.5	-	W24x76	Header	No Information Available	-	-
Floor 29	W14x398	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W18x97	Header	L3x3x5/16	11.5	-
Floor 28		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 27	W14x426	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 26		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 25	W14x455	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 24		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 23	W14x500	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W27x146	Header	L3x3x3/8	17.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 22		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 21	W14x550	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	17.5	-	W18x106	Header	L3x3x5/16	11.5	-
Floor 20		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 19	W14x605	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 18		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 17	W14x605	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 16		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 15	W14x665	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 14		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 13	W14x665	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 12		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 11	W14x730	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 10		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 9	W14x730	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x146	Header	L3x3x3/8	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 8		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 7	W14x730	W16x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Truss Cnx	-	-	-	W36x300	Header Pl	Pl 9x5/8	43	-	W36x300	Header Pl	Pl 9x5/8	24.5	-
Floor 6		-	-	-	-	-	W24x76	Fin	Pl 4.25x1/2	15.25	-	W36x210	Truss Cnx	-	-	-	W36x210	Truss Cnx	-	-	-
Floor 5	W14x730	-	-	-	-	-	W24x76	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-
Floor 4		-	-	-	-	-	W24x76	Fin	Pl 4.25x1/2	15	-	W36x260	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-
Floor 3	Sits on existing Col	-	-	-	-	-	W24x76	Fin	Pl 11x1	24.5	-	-	Truss Cnx	-	-	-	-	Truss Cnx	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.11 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 68

COLUMN 68		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W18x35	Knife	L4x3x3/8	8.5	0.58	W14x22	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L3x3x5/16	11.5	-	W33x130	Header	L3x3x5/16	11.5	-
Floor 47	W14x90	W36x150	Knife	L4x4x3/8	29.5	2.03	W24x55	Knife	L4x3x3/8	17.5	1.21	W24x146	Header	L3x3x5/16	14.5	-	W36x194	Header	L3x3x5/16	17.5	-
Floor 46		W16x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x84	Header	L3x3x5/16	14.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 45	W14x99	W18x35	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W30x132	Header	L3x3x5/16	20.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 44		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x68	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 43	W14x109	W14x22	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 42		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x68	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 41	W14x132	W14x22	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 40		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x68	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 39	W14x159	W14x22	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x162	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 38		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.59	W18x76	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 37	W14x176	W36x280	Seated	-	-	-	W12x19	Knife	L4x3x3/8	8.5	0.59	W36x150	Header	L3x3x5/16	29.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 36		W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 35	W14x193	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	No Information Available	-	-
Floor 34		W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W21x57	Header	No Information Available	-	-
Floor 33	W14x211	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	No Information Available	-	-
Floor 32		W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W21x57	Header	No Information Available	-	-
Floor 31	W14x233	W16x26	Knife	L4x3x3/8	8.5	0.58	W14x22	Knife	L4x3x3/8	8.5	0.58	W30x132	Header	L3x3x5/16	20.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 30		W33x118	Knife	L4x4x3/8	26.5	1.83	W33x130	Knife	L4x4x3/8	26.5	1.83	W36x160	Header	L3x3x5/16	29.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 29	W14x257	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 28		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 27	W14x257	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 26		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 25	W14x283	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 24		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	--	W21x57	Header	L3x3x5/16	11.5	-
Floor 23	W14x283	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 22		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 21	W14x283	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 20		W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 19	W14x311	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 18		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 17	W14x311	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 16		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 15	W14x342	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 14		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	MG-141	Header	L3x3x5/16	8.5		MG-141	Header	L3x3x5/16	8.5	
Floor 13	W14x342	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 12		W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 11	W14x370	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 10		W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 9	W14x370	W14x26	Knife	L4x3x3/8	8.5	0.59	W12x19	Knife	L4x3x3/8	8.5	0.59	W24x162	Header	L3x3x3/8	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 8		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.59	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 7	W14x665	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	W24x162	Header	L3x3x5/16	14.5	-	MG-28	Header	L3x3x5/16	8.5	-
Floor 6		W24x76	Fin	1Pl 4.25x0.5	15.25	-	W24x76	Fin	1Pl 4.25x1/2	21.25	-	W24x62	Header	L3x3x5/16	12.5	-	W24x68	Header	L3x3x5/16	12.5	-
Floor 5	W14x730	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	W30x150	Header	L3x3x5/16	29.5	-	W30x150	Header	L3x3x5/16	29.5	-
Floor 4		W24x76	Fin	1Pl 4.25x0.5	15.25	-	W24x76	Fin	1Pl 4.25x1/2	21	-	W8x31	Header	L3x3x5/16	8.5	-	W8x31	Header	L3x3x5/16	8.5	-
Floor 3	W14x730	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	MG-31	Header	L3x3x5/16	23.5	-	MG-31	Header	L3x3x5/16	23.5	-
Floor 2		W24x76	Fin	1Pl 4.25x0.5	21	-	W24x76	Fin	1Pl 7.25x3/4	18	-	W30x99	Header	L3x3x5/16	17.5	-	W30x99	Header	L3x3x5/16	17.5	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.12 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 69

COLUMN 69		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W14x22	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 47	W14x90	W24x55	Knife	L4x3x3/8	17.5	1.21	-	-	-	-	-	W24x62	Header	L3x3x5/16	14.5	-	W27x94	Header	L3x3x5/16	17.5	-
Floor 46		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W36x150	Header	L3x3x5/16	23.5	-	W33x130	Header	L3x3x5/16	20.5	-
Floor 45	W14x99	W14x22	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W27x84	Header	L3x3x5/16	14.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 44		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 43	W14x109	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	No Information Available	-	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 42		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	No Information Available	-	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 41	W14x132	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	No Information Available	-	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 40		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	No Information Available	-	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 39	W14x159	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 38		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 37	W14x193	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 36		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 35	W14x211	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 34		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 33	W14x233	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 32		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 31	W14x257	W14x22	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x76	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 30		W33x130	Knife	L4x4x3/8	26.5	1.83	-	-	-	-	-	W33x130	Header	L3x3x5/16	23.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 29	W14x283	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 28		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 27	W14x311	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 26		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 25	W14x342	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 24		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W18x71	Header	L3x3x5/16	11.5	-
Floor 23	W14x370	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 22		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 21	W14x398	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 20		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 19	W14x426	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 18		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 17	W14x455	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W18x71	Header	L3x3x5/16	11.5	-
Floor 16		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 15	W14x500	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 14		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	MG-142	Header	L3x3x5/16	8.5	-	MG-142	Header	L3x3x5/16	8.5	-
Floor 13	W14x500	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 12		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 11	W14x550	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 10		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 9	W14x550	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 8		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W21x68	Header	L3x3x5/16	11.5	-
Floor 7	W14x730	W24x76	Truss Cnx	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.59	W36x210	Header	Pl7x5/8	21	-	W36x210	Header	Pl 7x5/8	21	-
Floor 6		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	W36x210	Truss Cnx	-	-	-	W36x210	Truss Cnx	-	-	-
Floor 5	W14x730 + 2Web Pl's 12.5x3	W24x76	Truss Cnx	-	-	-	-	-	-	-	-	W36x300	Truss Cnx	-	-	-	W36x300	Truss Cnx	-	-	-
Floor 4		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	W8x31	Truss Cnx	-	-	-	W8x31	Truss Cnx	-	-	-
Floor 3	W14x730 + 2Web Pl's 12.5x4	W24x76	Truss Cnx	-	-	-	-	-	-	-	-	W36x300	Truss Cnx	-	-	-	W36x300	Truss Cnx	-	-	-
Floor 2		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	W36x300	Truss Cnx	-	-	-	W36x300	Truss Cnx	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.13 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 70

COLUMN 70		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	-	-	-	-	-	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	17.5	-	W30x116	Header	L3x3x5/16	14.5	-
Floor 47	W14x99	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x68	Header	L3x3x5/16	14.5	-	W24x146	Header	L3x3x5/16	14.5	-
Floor 46		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	MG-61	Header	L4x3x7/16	22	-	W36x230	Header	L4x3x1/2	24	-
Floor 45	W14x132	-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	20.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 44		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	20.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 43	W14x159	-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	No Information Available	-	-	W27x146	Header	L3x3x5/16	17.5	-
Floor 42		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	20.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 41	W14x193	-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	No Information Available	-	-	W27x146	Header	L3x3x5/16	17.5	-
Floor 40		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	20.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 39	W14x257	-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	20.5	-	W27x146	Header	L3x3x5/16	17.5	-
Floor 38		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	20.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 37	W14x342	-	-	-	-	-	Girder MG371	Knife	L4x4x3/8	20.5	1.28	40' Girder	Header	L3x3x5/16	20.5	-	W36x170	Header	L3x3x3/8	38.5	-
Floor 36		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 35	W14x370	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 34		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 33	W14x398	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 32		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 31	W14x426	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W27x146	Header	L3x3x3/8	17.5	-
Floor 30		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W27x94	Header	L3x3x5/16	14.5	-
Floor 29	W14x455	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 28		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 27	W14x500	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 26		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 25	W14x550	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	20.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 24		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W27x84	Header	L3x3x5/16	17.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 23	W14x605	-	-	-	-	-	W18x35	Knife	L4x3x3/8	11.5	0.79	W30x108	Header	L3x3x5/16	17.5	-	W27x146	Header	L3x3x3/8	17.5	-
Floor 22		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	14.5	-
Floor 21	W14x605	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L3x3x5/16	14.5	-	W27x146	Header	L3x3x3/8	17.5	-
Floor 20		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 19	W14x665	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 18		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 17	W14x665	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 16		-	-	-	-	-	W18x35	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L3x3x5/16	17.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 15	W14x730	-	-	-	-	-	W36x280	Seated	-	-	-	W36x170	Seated	-	-	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 14		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 13	W14x730	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 12		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 11	W14x730 + 2Web Pl's 12.5x1	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 10		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 9	W14x730 + 2Web Pl's 12.5x1	-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x146	Header	L3x3x3/8	14.5	-
Floor 8		-	-	-	-	-	W14x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 7	W14x730 + 2Web Pl's 12.5x1.5	W16x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Truss Cnx	-	-	-	W36x300	Double Fin	Pl 4.5x1/2	24.5	-	W36x300	Double Fin	Pl 4.5x1/2	49.5	-
Floor 6		-	-	-	-	-	W24x76	Fin	Pl 4.25x1/2	15.25		W36x210	Truss Cnx	-	-	-	W36x210	Truss Cnx	-	-	-
Floor 5	W14x730 + 2Web Pl's 12.5x1.5	-	-	-	-	-	W24x76	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-
Floor 4		-	-	-	-	-	W24x76	Fin	Pl 4.25x1/2	15	-	W36x260	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-
Floor 3	Sits on Existing Col	-	-	-	-	-	W24x76	Fin	Pl 11x1	24.5	-	-	Truss Cnx	-	-	-	-	Truss Cnx	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.14 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 71

COLUMN 71		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W16x31	Knife	L4x3x3/8	8.5	0.58	W14x22	Knife	L4x3x3/8	8.5	0.58	W24x68	Header	L3x3x5/16	11.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 47	W14x90	W21x50	Knife	L4x3x3/8	14.5	1.00	W18x35	Knife	L4x3x3/8	11.5	0.78	W30x99	Header	L3x3x5/16	23.5	-	W24x146	Header	L3x3x5/16	14.5	-
Floor 46		W24x55	Knife	L4x3x3/8	17.5	1.21	W18x35	Knife	L4x3x3/8	11.5	0.78	W30x99	Header	L3x3x5/16	14.5	-	W24x84	Header	L3x3x5/16	17.5	-
Floor 45	W14x99	W24x55	Knife	L4x3x3/8	17.5	1.21	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W30x132	Header	L3x3x5/16	20.5	-
Floor 44		W24x55	Knife	L4x3x3/8	17.5	1.21	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 43	W14x132	W24x55	Knife	L4x3x3/8	17.5	1.21	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x5/16	14.5	-
Floor 42		W24x55	Knife	L4x3x3/8	17.5	1.21	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 41	W14x159	W24x55	Knife	L4x3x3/8	17.5	1.21	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x5/16	14.5	-
Floor 40		W24x55	Knife	L4x3x3/8	17.5	1.21	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 39	W14x193	W24x55	Knife	L4x3x3/8	18.5	1.28	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x5/16	14.5	-
Floor 38		W24x55	Knife	L4x3x3/8	18.5	1.28	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W18x76	Header	L3x3x5/16	11.5	-
Floor 37	W14x233	Girder MG371	Knife	L4x4x3/8	20.5	1.41	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W36x150	Header	L3x3x5/16	29.5	-
Floor 36		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 35	W14x257	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 34		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 33	W14x283	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 32		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 31	W14x283	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W30x132	Header	L3x3x5/16	20.5	-
Floor 30		W21x50	Knife	L4x3x3/8	14.5	1.00	W36x150	Knife	L4x4x3/8	29.5	2.03	W27x94	Header	L3x3x5/16	14.5	-	W36x160	Header	L3x3x5/16	29.5	-
Floor 29	W14x342	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 28		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 27	W14x370	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 26		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 25	W14x398	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x94	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 24		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 23	W14x426	W14x22	Knife	L4x3x3/8	11.5	0.79	W12x19	Knife	L4x4x3/8	26.5	1.83	W36x160	Header	L3x3x5/16	29.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 22		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 21	W14x426	W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W27x84	Header	L3x3x5/16	14.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 20		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 19	W14x455	W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 18		W14x22	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W24x76	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 17	W14x455	W18x35	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W18x97	Header	L3x3x5/16	11.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 16		W18x35	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W30x108	Header	L3x3x5/16	17.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 15	W14x500	W36x280	Seated	-	-	-	W12x19	Knife	L4x3x3/8	8.5	0.58	Girder MG151	Header	L3x3x5/16	23.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 14		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	MG-141	Header	L3x3x5/16	8.5	-
Floor 13	W14x500	W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 12		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 11	W14x500	W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 10		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 9	W14x550	W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W24x162	Header	L3x3x3/8	14.5	-
Floor 8		W14x26	Knife	L4x3x3/8	8.5	0.58	W12x19	Knife	L4x3x3/8	8.5	0.58	W21x57	Header	L3x3x5/16	11.5	-	W21x57	Header	L3x3x5/16	11.5	-
Floor 7	W14x665	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	W24x76	Header	L3x3x5/16	11.5	-	W24x162	Header	L3x3x5/16	14.5	-
Floor 6		W24x76	Fin	Pl 4.25x1/2	15.25		W24x76	Fin	Pl 4.25x1/2	21.25		W21x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	12.5	-
Floor 5	W14x730	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	W36x150	Header	L3x3x5/16	29.5	-	W36x150	Header	L3x3x5/16	29.5	-
Floor 4		W24x76	Fin	Pl 4.25x1/2	15.25		W24x76	Fin	Pl 4.25x1/2	21		W33x130	Header	L3x3x5/16	23.5	-	W8x31	Header	L3x3x5/16	5.5	-
Floor 3	W14x730	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	MG-31	Header	L3x3x1/4	23.5	-	Girder MG31	Header	L3x3x5/16	23.5	-
Floor 2		W24x76	Fin	Pl 4.25x1/2	21		W24x76	Fin	Pl 7.25x3/4	18		W21x50	Header	L3x3x5/16	11.5	-	W30x99	Header	L3x3x5/16	17.5	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.15 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 72

COLUMN 72																					
		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W14x22	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W21x73	Header	3x3x5/16	11.5	-	W24x62	Header	3x3x5/16	11.5	-
Floor 47	W14x99	W18x35	Knife	L4x3x3/8	11.5	0.79	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	3x3x5/16	14.5	-
Floor 46		W18x35	Knife	L4x3x3/8	11.5	0.79	-	-	-	-	-	Girder MG61	Header	4x3x7/16	22	-	W36x150	Header	4x3x3/8	18	-
Floor 45	W14x120	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 44		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 43	W14x132	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	No Information Available	-	-	W24x68	Header	No Information Available	-	-
Floor 42		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	No Information Available	-	-
Floor 41	W14x159	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	No Information Available	-	-	W24x68	Header	No Information Available	-	-
Floor 40		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	No Information Available	-	-
Floor 39	W14x193	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 38		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 37	W14x233	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 36		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 35	W14x283	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 34		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 33	W14x311	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 32		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 31	W14x342	W16x31	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 30		W36x150	Knife	L4x4x3/8	29.5	2.03	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W33x130	Header	L3x3x5/16	23.5	-
Floor 29	W14x370	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 28		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 27	W14x398	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 26		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 25	W14x426	W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 24		W16x26	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W28x71	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 23	W14x455	W33x130	Knife	L4x4x3/8	23.5	1.62	-	-	-	-	-	W33x130	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 22		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 21	W14x500	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 20		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 19	W14x550	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 18		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 17	W14x550	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W18x71	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 16		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 15	W14x605	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 14		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	MG-142	Header	L3x3x5/16	8.5	-
Floor 13	W14x605	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 12		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 11	W14x665	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 10		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 9	W14x665	W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 8		W12x19	Knife	L4x3x3/8	8.5	0.59	-	-	-	-	-	W24x62	Header	L3x3x5/16	11.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 7	W14x730	W24x76	Truss Cnx	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.59	W36x210	Header	Pl7x5/8	21	-	W36x210	Header	Pl7x5/8	21	-
Floor 6		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	W36x210	Truss Cnx	-	-	-	W36x210	Truss Cnx	-	-	-
Floor 5	W14x730 + 2Web Pl's 12.5x1.5	W24x76	Truss Cnx	-	-	-	-	-	-	-	-	W36x300	Truss Cnx	-	-	-	W36x300	Truss Cnx	-	-	-
Floor 4		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	W8x31	Truss Cnx	-	-	-	W8x31	Truss Cnx	-	-	-
Floor 3	W14x730 + 2Web Pl's 12.5x3	W24x76	Truss Cnx	-	-	-	-	-	-	-	-	W36x300	Truss Cnx	-	-	-	W36x300	Truss Cnx	-	-	-
Floor 2		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	MG-29	Truss Cnx	-	-	-	W36x300	Truss Cnx	-	-	-

Notes
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
2. Where no beams size is indicated no member is framing into the column
3. The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.16 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 73

COLUMN 73		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x99	W33x118	Knife	L4x3x3/8	17.5	1.21	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W27x84	Header	L3x3x5/16	14.5	-
Floor 47	W14x109	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x62	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	14.5	-
Floor 46		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	W36x210	Header	L4x3x1/2	30.5	-	Girder MG61	Header	L4x3x7/16	22	-
Floor 45	W14x145	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 44		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 43	W14x176	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 41	W14x211	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 39	W14x257	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 38		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 37	W14x342	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 36		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 35	W14x398	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 34		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 33	W14x455	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5		40' Girder	Header	L3x3x5/16	20.5	-
Floor 32		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 31	W14x500	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 30		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 29	W14x550	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 28		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 27	W14x605	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 26		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 25	W14x665	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	W24x76	Header	L3x3x5/16	17.5	-	40' Girder	Header	L3x3x5/16	20.5	-
Floor 24		-	-	-	-	-	W24x76	Knife	L4x3x3/8	17.5	1.21	40' Girder	Header	L3x3x5/16	23.5	-	W27x84	Header	L3x3x5/16	17.5	-
Floor 23	W14x730	-	-	-	-	-	Girder MG371	Knife	L4x3x3/8	20.5	1.41	40' Girder	Header	L3x3x5/16	23.5	-	W27x94	Header	L3x3x5/16	17.5	-
Floor 22		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Header	L3x3x5/16	23.5	-	W27x94	Header	L3x3x5/16	14.5	-
Floor 21	W14x730 + 2Web Pl's 12.5x1	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00
Floor 20		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Seated	-	-	-	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 19	W14x730 + 2Web Pl's 12.5x2	-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Seated	-	-	-	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 18		-	-	-	-	-	W21x50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Seated	-	-	-	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 17	W14x730 + 2Web Pl's 12.5x3	-	-	-	-	-	W21 50	Knife	L4x3x3/8	14.5	1.00	40' Girder	Seated	-	-	-	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 16		-	-	-	-	-	W24x55	Knife	L4x3x3/8	17.5	1.21	40' Girder	Seated	-	-	-	W30x99	Knife	L4x3x3/8	17.5	1.21
Floor 15	W14x730 + 2Web Pl's 12.5x4	-	-	-	-	-	Girder MG152	Knife	L4x3x3/8	26.5	1.83	40' Girder	Seated	-	-	-	W24x76	Seated	-	-	-
Floor 14		-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 13	W14x730 + 2Web Pl's 12.5x5	-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 12		-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 11	W14x730 + 2Web Pl's 12.5x6	-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 10		-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 9	W14x730 + 2Web Pl's 12.5x6.5	-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 8		-	-	-	-	-	W16x40	Knife	L4x3x3/8	8.5	0.59	40' Girder	Seated	-	-	-	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 7	W14x730 + 2Web Pl's 12.5x8	W16x26	Knife	L4x3x3/8	8.5	0.59	W24x76	Truss Cnx	-	-	-	W36x210	Knife	L4x4x1/2 - 3/8 weld	46.5	3.85	W36x300	Double Fin	Pl 4.5x1/2	24.5	1.69
Floor 6		None	Fin	Pl 10x3/4	18	-	W24x76	Fin	Pl 4.5x1/2	15.25	-	W33x118	No Info	-	-	-	W36x210	Truss Cnx	-	-	-
Floor 5	W14x730 + 2Web Pl's 12.5x8	-	-	-	-	-	W24x76	Truss Cnx	-	-	-	TRUSS #1	Truss Cnx	-	-	-	W36x260	Truss Cnx	-	-	-
Floor 4		-	-	-	-	-	W24x76	Fin (+Angle)	Pl 4.5x1/2 (L5x5x3/8)	15 (18)	-	W36x194	Knife	L4x3x3/8	29.5	2.03	W36x260	Truss Cnx	-	-	-
Floor 3	Existing Col	-	-	-	-	-	W24x76	Fin	Pl 11x1	24.5	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.17 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 74

COLUMN 74

		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W14x22	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x49	Header	L4x3x5/16	14.5	-	W24x68	Header	L4x3x5/16	11.5	-
Floor 47	W14x90	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x76	Header	L4x3x5/16	14.5	-	W30x99	Header	L4x3x5/16	23.5	-
Floor 46		W24x55	Knife	L4x3x3/8	17.5	1.21	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16	14.5	-	W30x99	Header	L4x3x5/16	14.5	-
Floor 45	W14x99	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 L4x3x5/16	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 44		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 L4x3x5/16	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 43	W14x132	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 41	W14x176	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 39	W14x211	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 38		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 37	W14x257	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 36		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 35	W14x342	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 34		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 33	W14x370	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 32		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 31	W14x398	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 30		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 29	W14x426	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 28		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 27	W14x455	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 26		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 25	W14x500	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16	14.5	-
Floor 24		W24x76	Knife	L4x3x3/8	17.5	1.21	W16x31	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L5x3x5/16 bent-L4x3x5/16 bent	23.5	-	W27x84	Header	L4x3x5/16	14.5	-
Floor 23	W14x550	Girder MG371	Knife	L4x4x3/8	20.5	1.41	W36x150	Knife	L4x4x3/8	29.5	2.03	W27x84	Header	L5x3x3/8 bent-L4x3x3/8 bent	23.5	-	W27x84	Header	L4x3x3/8	29.5	-
Floor 22		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L4x3x5/16	14.5	-
Floor 21	W14x550	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L4x3x5/16	14.5	-
Floor 20		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x76	Header	L4x3x5/16	11.5	-
Floor 19	W14x605	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x76	Header	L4x3x5/16	11.5	-
Floor 18		W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x76	Header	L4x3x5/16	11.5	-
Floor 17	W14x605	W21x50	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W18x97	Header	L4x3x5/16	11.5	-
Floor 16		W24x55	Knife	L4x3x3/8	18.5	1.28	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W30x108	Header	L4x3x5/16	17.5	-
Floor 15	W14x665	Girder MG152	Knife	L4x4x3/8	26.5	1.83	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	Girder MG151	Header	L4x3x5/16	23.5	-
Floor 14		W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 13	W14x665	W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 12		W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 11	W14x730	W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 10		W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 9	W14x730	W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 8		W16x40	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W21x57	Header	L4x3x5/16	11.5	-
Floor 7	W14x730 + 2Web Pl's 12.5x6.5	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	W33x118	Knife	L4x3x3/8	20.5	1.41	W24x76	Knife	L4x3x3/8	11.5	0.79
Floor 6		W24x76	Double Fin	Pl 4.25x1/2	18.25	-	W24x76	Fin	Pl 4.25x1/2	21.25	-	W21x62	-	-	-	-	W21x62	Knife	L4x3x3/8	11.5	0.79
Floor 5	W14x730 + 2Web Pl's 12.5x8	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	GIRDER 48	Truss Cnx	-	-	-	W30x150	Knife	L4x4x3/8	29.5	2.03
Floor 4		W24x76	Fin (+Angle)	Pl 4.25x1/2 (L5x5x3/8)	15.25 (18)	-	W24x76	Fin (+Angle)	Pl 4.25x1/2 (L5x5x3/8)	21.25 (12)	-	-	-	-	-	-	W33x130	Knife	L4x4x3/8	23.5	1.62
Floor 3	W14x730 + 2Web Pl's 12.5x8	W24x76	Truss Cnx	-	-	-	W24x76	Truss Cnx	-	-	-	-	-	-	-	-	MG-31	Knife	L4x4x3/8	23.5	1.62
Floor 2		W24x76	Fin (+Angle)	Pl 4.25x1/2 (L5x5x3/8)	21 (18)	-	W24x76	Fin (+Angle)	Pl 7.25x3/4 (L5x5x3/8)	18 (12)	-	-	-	-	-	-	W21x50	Knife	L4x3x3/8	11.5	0.79

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.18 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 75

COLUMN 75

		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W14x22	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x76	Header	L3x3x5/16	11.5		W21x73	Header	L3x3x5/16	11.5	-
Floor 47	W14x99	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x76	Header	L3x3x5/16	14.5		40' Girder	Header	No Information Available	-	-
Floor 46		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W36x150	Header	L4x3x1/2	18		MG-61	Header	L4x3x7/16	22	-
Floor 45	W14x109	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5		40' Girder	Header	L3x3x5/16	23.5	-
Floor 44		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5		40' Girder	Header	L3x3x5/16	23.5	-
Floor 43	W14x132	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 41	W14x176	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 39	W14x233	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 38		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 37	W14x283	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 36		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 35	W14x342	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 34		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 33	W14x370	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 32		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 31	W14x398	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 30		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 29	W14x426	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 28		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 27	W14x455	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 26		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 25	W14x500	W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	W24x68	Header	L3x3x5/16	11.5	-	40' Girder	Header	L3x3x5/16	23.5	-
Floor 24		W16x31	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W18x71	Header	L3x3x5/16	11.5	-
Floor 23	W14x550	W30x150	Knife	L4x4x3/8	29.5	2.03	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W33x130	Header	L3x3x5/16	23.5	-
Floor 22		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 21	W14x550	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 20		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 19	W14x605	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 18		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 17	W14x605	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W18x71	Header	L3x3x5/16	11.5	-
Floor 16		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 15	W14x665	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 14		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 13	W14x665	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 12		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 11	W14x730	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 10		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 9	W14x730	W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 8		W16x26	Knife	L4x3x3/8	8.5	0.58	-	-	-	-	-	40' Girder	Header	L3x3x5/16	23.5	-	W24x62	Header	L3x3x5/16	11.5	-
Floor 7	W14x730	W24x76	Truss Cnx	-	-	-	W16x26	Knife	L4x3x3/8	8.5	0.58	W27x102	Header	L3x3x5/16	20.5	-	W36x210	Header	Pl7x5/8	21	-
Floor 6		W24x76	Fin	Pl 4.25x1/2	21	-	-	-	-	-	-	W33x130	Header	L3.5x3x3/8	24	-	W36x210	Truss Cnx	-	-	-
Floor 5	W14x730 + 2Web Pl's 12.5x1.5	W24x76	Truss Cnx	-	-	-	-	-	-	-	-	W33x201	Knife	L4x3x1/2	26	1.79	W36x300	Truss Cnx	-	-	-
Floor 4		W24x76	Fin (+Angle)	Pl 4.5x1/2 (L5x5x3/8)	21 (12)	-	-	-	-	-	-	-	-	-	-	-	W8x31	Truss Cnx	-	-	-
Floor 3	W14x730 + 2Web Pl's 12.5x3	W24x76	Truss Cnx	-	-	-	W30x108	Knife	L4x3x3/8	14.5	1.00	W16x26	Knife	L4x3x3/8	8.5	0.58	W36x200	Truss Cnx	-	-	-
Floor 2		W24x76	Fin (+Angle)	Pl 7.5x3/4	18	-	-	-	-	-	-	-	-	-	-	-	MG-29	Truss Cnx	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.19 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 76

COLUMN 76		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W27x84	Header (Offset)	L4x3x5/16	14.5	-	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x160	Header	L4x3x5/16 bent	26.5	-	W27x84	Header	L4x3x5/16	17.5	-
Floor 47	W14x120	W14x30	Header (Offset)	L5x3x5/16	8.5	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W35x135	Header	L4x3x5/16 bent	21.5	-	W24x62	Header	L3x3x5/16	14.5	-
Floor 46		W36x210	Header (Offset)	L4x3x3/8	14.5	-	W24x55	Knife	L4x4x3/8	14.5	1.00	W36x150	Header	L4x4x3/8 bent	21	-	W36x210	Header	L4x3x1/2	30.5	-
Floor 45	W14x159	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W35x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 44		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W35x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 43	W14x193	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 41	W14x257	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 39	W14x311	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 38		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 37	W14x370	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	24	1.66	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 36		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	24	1.66	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 35	W14x426	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 34		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x3x3/8	14.5	1.00
Floor 33	W14x455	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 32		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 31	W14x500	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 30		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 29	W14x550	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 28		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 27	W14x606	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 26		-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 25	W14x665	-	-	-	-	-	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	W24x76	Knife	L4x4x3/8	14.5	1.00
Floor 24		W24x55	Knife	L4x3x3/8	11.5	0.79	W14x22	Knife	L4x3x3/8	8.5	0.58	W36x135	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 23	W14x730	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x150	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 22		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x150	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 21	W14x730 + 2 WEB Pl 12.5x1	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x150	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 20		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 19	W14x730 + 2 WEB Pl 12.5x2	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 18		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 17	W14x730 + 2 WEB Pl 12.5x3	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 16		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x4x3/8	23.5	1.62	40' Girder	Seated	-	-	-
Floor 15	W14x730 + 2 WEB Pl 12.5x4	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 14		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 13	W14x730 + 2 WEB Pl 12.5x5	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 12		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 11	W14x730 + 2 WEB Pl 12.5x6	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 10		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 9	W14x730 + 2 WEB Pl 12.5x8	W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 8		W24x55	Knife	L4x3x3/8	11.5	0.79	W21x50	Knife	L4x3x3/8	11.5	0.79	W36x135	Knife	L4x3x3/8	24	1.66	40' Girder	Seated	-	-	-
Floor 7	Sits on Truss 1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.20 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 77

COLUMN 77

		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W21x50	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16 bent	14.5	-
Floor 47	W14x90	W10x45	Knife	L4x3x3/8	14.5	1.00	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x76	Header	L4x3x5/16 bent	14.5	-
Floor 46		W24x55	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x94	Header	L4x3x5/16 bent	14.5	-
Floor 45	W14x99	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 44		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 43	W14x120	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 41	W14x145	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 39	W14x176	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 38		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 37	W14x233	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 36		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 35	W14x283	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 34		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 33	W14x342	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 32		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 31	W14x370	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 30		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 29	W14x398	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 28		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 27	W14x426	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 26		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 25	W14x455	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W24x68	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 24		W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L4x3x5/16 bent	14.5	-	W30x99	Header	L5x3x5/16 bent-L4x3x5/16 bent	23.5	-
Floor 23	W14x500	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x3/8 bent-L4x3x3/8 bent	23.5	-
Floor 22		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 21	W14x550	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W30x99	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 20		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 19	W14x605	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 18		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 17	W14x605	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 16		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 15	W14x665	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 14		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 13	W14x665	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 12		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 11	W14x730	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 10		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 9	W14x730	W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 8		W21x50	Knife	L4x3x3/8	11.5	0.79	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x94	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 7	Sits on Truss 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.21 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 78

COLUMN 78		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W14x22	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W24x68	Header	L3x3x5/16	11.5	-	W24x76	Header	L3x3x5/16	11.5	-
Floor 47	W14x90	W16x31	Knife	L4x3x3/8	8.5	0.59	W14x22	Knife	L4x3x3/8	8.5	0.59	W24x84	Header	L3x3x5/16	14.5	-	W24x76	Header	L3x3x5/16	14.5	-
Floor 46		W14x22	Knife	L4x3x3/8	11.5	0.79	W21x44	Knife	L4x3x3/8	8.5	0.59	W33x130	Header	L3x3x5/16	20.5	-	W36x150	Header	L3x3x5/16	20.5	-
Floor 45	W14x99	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 44		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 43	W14x120	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	No Information Available	-	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 41	W14x145	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	No Information Available	-	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 39	W14x176	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	No Information Available	-	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 38		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	No Information Available	-	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 37	W14x233	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 36		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 35	W14x283	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 34		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 33	W14x342	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 32		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 31	W14x370	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 30		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 29	W14x398	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 28		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 27	W14x426	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 26		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 25	W14x455	W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	W24x68	Header	L3x3x5/16	11.5	-
Floor 24		W14x22	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 23	W14x500	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 22		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 21	W14x550	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x31	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 20		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 19	W14x605	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 18		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 17	W14x605	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 16		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 15	W14x665	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 14		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 13	W14x665	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 12		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 11	W14x730	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 10		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 9	W14x730	W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 8		W16x31	Knife	L4x3x3/8	8.5	0.59	W16x26	Knife	L4x3x3/8	8.5	0.59	W27x84	Header	L3x3x5/16	14.5	-	GIRDER40	Header	L3x3x5/16	23.5	-
Floor 7	Sits on Transfer	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.22 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 79

COLUMN 79		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x99	W36x150	Header	L4x3x3/8 bent L5x3x3/8 bent	26.5	-	W30x99	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x160	Header Pl	9x3/8	27	-
Floor 47	W14x132	W33x130	Seated	-	-	-	W24x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Header Pl	9x3/8	24	-
Floor 46		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x150	Knife	8.5x3/8 bent	24.5	1.69
Floor 45	W14x193	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 44		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 43	W14x257	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 41	W14x311	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 39	W14x370	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 38		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 37	W14x426	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 36		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 35	W14x455	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 34		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 33	W14x500	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 32		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 31	W14x550	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 30		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 29	W14x605	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 28		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 27	W14x665	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 26		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 25	W14x730	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 24		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 23	W14x730 + 2 SIDE Pl 26x1	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x150	Knife	8.5x3/8 bent	24.5	1.69
Floor 22		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x150	Knife	8.5x3/8 bent	24.5	1.69
Floor 21	W14x730 + 2 SIDE Pl 26x1	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x150	Knife	8.5x3/8 bent	24.5	1.69
Floor 20		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 19	W14x730 + 2 SIDE Pl 26x1	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 18		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 17	W14x730 + 2 SIDE Pl 26x2	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 16		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 15	W14x730 + 2 SIDE Pl 26x2	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 14		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 13	W14x730 + 2 SIDE Pl 26x2	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 12		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 11	W14x730 + 2 SIDE Pl 26x3	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 10		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 9	W14x730 + 2 SIDE Pl 26x3	W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 8		W33x130	Seated	-	-	-	W27x94	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W36x135	Knife	8.5x3/8 bent	24.5	1.69
Floor 7	W14x730 + 2 SIDE Pl's 26x3	W30x173	Knife	L4x3x3/8	23.5	1.62	W36x160	Knife	L4x4x1/2	30.5	2.10	-	-	-	-	-	W24x146	Knife	L4x3x3/8	11.5	0.79
Floor 6		W27x146	Knife	L4x3x3/8	17.5	1.21	W24x104	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W21x30	Knife	7x3/8 bent 6.75x3/8 bent	12	0.83
Floor 5	W14x730 + 2 SIDE Pl's 26x4	MG-54	Knife	L4x4x1/2-7/16 weld	21.5	2.08	W30x211	Knife	L4x4x1/2-7/16 weld	21.5	2.08	-	-	-	-	-	W33x152	Knife	8x3/8 bent	27	1.86
Floor 4		W36x230	Knife	L4x4x1/2-7/16 weld	30	2.90	W30x173	Knife	L4x4x1/2-7/16 weld	24	2.32	-	-	-	-	-	W30x108	Knife	8x3/8 bent	18	1.24
Floor 3	W14x730 + 2 SIDE Pl's 26x6	W36x245	Knife	L5x3x1/2-7/16 weld	21	2.03	W36x150	Knife	L5x3x3/8	24	1.66	W12x19	Knife	L4x3x3/8	12	0.83	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.23 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 80

COLUMN 80		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W30x99	Knife	L4x3x3/8	14.5	1.00	W24x68	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W30x99	Header Pl	9x3/8	15	-
Floor 47	W14x99	W24x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 46		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 45	W14x145	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 44		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 43	W14x193	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 41	W14x233	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 39	W14x283	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 38		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 37	W14x322	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 36		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 35	W14x398	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 34		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 33	W14x455	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 32		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 31	W14x500	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 30		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 29	W14x550	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 28		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 27	W14x605	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 26		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 25	W14x665	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 24		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 23	W14x730	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x68	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W30x99	Knife	7.5x3/8 bent	18.5	1.28
Floor 22		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x68	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W30x99	Knife	7.5x3/8 bent	18.5	1.28
Floor 21	W14x730 + 2 WEB Pl 12.5x1	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x68	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W30x99	Knife	7.5x3/8 bent	18.5	1.28
Floor 20		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 19	W14x730 + 2 WEB Pl 12.5x2	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 18		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 17	W14x730 + 2 WEB Pl 12.5x3	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 16		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 15	W14x730 + 2 WEB Pl 12.5x4	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 14		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 13	W14x730 + 2 WEB Pl 12.5x5	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 12		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 11	W14x730 + 2 WEB Pl 12.5x6	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 10		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Knife	7.5x3/8 bent	18.5	1.28
Floor 9	W14x730 + 2 WEB Pl 12.5x8	W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Seated	-	-	-
Floor 8		W27x94	Knife	L4x3x3/8	14.5	1.00	W24x62	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	W27x94	Seated	-	-	-
Floor 7	W14x730 + 2 WEB Pl 12.5x8	W36x160	Knife	L4x4x3/8	26.5	1.83	W30x99	Knife	L4x4x3/8	23.5	1.62	-	-	-	-	-	TRUSS #2	Knife	7.5x3/8 bent	21	1.45
Floor 6		W24x104	Knife	L4x3x3/8	14.5	1.00	-	-	-	-	-	-	-	-	-	-	TRUSS #2	Single Angle	7.5x3/8 bent	9	-
Floor 5	W14x730 + 2 WEB Pl 12.5x8	W30x211	Knife	L4x3x1/2-7/16 weld	21	2.03	W30x173	Knife	L4x3x1/2-7/16 weld	24	2.32	-	-	-	-	-	TRUSS #2	Truss Cnx	-	-	-
Floor 4		W30x173	M-Conn	-	-	-	W18x35	M-Conn	-	-	-	W30x211	Knife	6.5x1/2 bent 7/16 weld	24	2.32	W27x94	Knife	6.5x1/2 bent	18	1.24
Floor 3	W14x730 + 2 SIDE Pl 26x5	W36x150	Knife	L5x3x3/8	24	1.66	W36x135	Knife	L5x3x3/8	27	1.86	W12x19	Knife	L4x3x3/8	9	0.62	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C5.24 INTERIOR COLUMN CONNECTION TYPE CATALOGUE – COLUMN 81

COLUMN 81		North					South					East					West				
	Column	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]	Beam	Type	Size	Length [in]	Factor[1]
Roof	W14x90	W24x68	Knife	L4x3x3/8	14.5	1	-	-	-	-	-	W24x117	Header	L4x3x3/8 bent	11.5	-	W24x68	Header	L4x3x5/16	11.5	-
Floor 47	W14x99	W24x62	Knife	L4x3x3/8	11.5	0.79	-	-	-	-	-	W24x117	Header	L4x3x3/8 bent	11.5	-	W24x84	Header	L4x3x5/16 bent-L5x3x5/16 bent	14.5	-
Floor 46		W24x62	Knife	L4x3x3/8	11.5	0.79	-	-	-	-	-	W36x300	Header	L4x3x1/2 bent	24.5	-	W33x130	Header	L4x3x3/8 bent	18	-
Floor 45	W14x145	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L4x3x5/16 bent	11.5	-	W27x84	Header	L4x3x5/16 bent-L5x3x5/16 bent	14.5	-
Floor 44		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L4x3x5/16 bent	11.5	-	W27x84	Header	L4x3x5/16 bent-L5x3x5/16 bent	14.5	-
Floor 43	W14x176	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 42		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	No Information Available	-	-
Floor 41	W14x211	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 40		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	No Information Available	-	-
Floor 39	W14x257	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	No Information Available	-	-
Floor 38		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	No Information Available	-	-
Floor 37	W14x311	W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 36		W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 35	W14x370	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 34		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 33	W14x426	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 32		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 31	W14x455	W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 30		W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 29	W14x500	W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 28		W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 27	W14x550	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 26		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 25	W14x605	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 24		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 23	W14x665	W24x68	Knife	L4x3x3/8	14.5	1.00	W27x94	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 22		W24x68	Knife	L4x3x3/8	14.5	1.00	W27x94	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 21	W14x665	W24x68	Knife	L4x3x3/8	14.5	1.00	W27x94	Seated	-	-	-	W24x55	Header	L5x3x3/8 bent-L4x3x3/8 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 20		W24x62	Knife	L4x3x3/8	14.5	1.00	W27x94	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 19	W14x730	W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 18		W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 17	W14x730	W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 16		W24x62	Knife	L4x3x3/8	14.5	1.00	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 15	W14x730 + 2 WEB Pl 12.5x1	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 14		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 13	W14x730 + 2 WEB Pl 12.5x1	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 12		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Header	L5x3x5/16 bent-L4x3x5/16 bent	11.5	-	W27x84	Header	L5x3x5/16 bent-L4x3x5/16 bent	14.5	-
Floor 11	W14x730 + 2 WEB Pl 12.5x2	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Knife	L4x3x3/8	11.5	0.79	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 10		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Knife	L4x3x3/8	11.5	0.79	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 9	W14x730 + 2 WEB Pl 12.5x3	W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Knife	L4x3x3/8	11.5	0.79	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 8		W24x62	Knife	L4x3x3/8	11.5	0.79	W27x84	Seated	-	-	-	W24x55	Knife	L4x3x3/8	11.5	0.79	W27x84	Knife	L4x3x3/8	14.5	1.00
Floor 7	W14x730 + 2 WEB Pl's 12.5x3.5	W30x99	Knife	L4x4x3/8	23.5	1.62	MG-26	Double Fin	Pl 11x7/8	17	-	-	-	-	-	-	W27x114	Knife	L4x4x3/8	21	1.45
Floor 6		-	-	-	-	-	MG-61	Double Fin	Pl 8x5/8	16	-	W30x81	Knife	L4x3x3/8	15	1.03	W33x130	Knife	L4x4x3/8	21	1.45
Floor 5	W14x730 + 2 WEB Pl's 12.5x6.5	W30x173	Knife	L4x4x1/2-7/16 weld	23	2.22	MG-52	Double Fin	Pl 9.625x1	21.5	-	W27x84	Single Angle	L8x4x1/2	24	-	W33x201	Double Fin	Pl 4.375x5/8	27.5	-
Floor 4		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3	W14x730 + 2 WEB Pl's 12.5x4.5	W36x135	Knife	L4x3x3/8	27	1.86	W16x26	Knife	L4x3x3/8	11.5	0.79	MG-32	Knife	L4x3x3/8	30	2.07	-	-	-	-	-
Floor 2		W36x135	No Info	-	-	-	-	-	-	-	-	W36x300	Knife	L4x3x3/8	46	3.17	-	-	-	-	-

Notes
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
2 Where no beams size is indicated no member is framing into the column
3 The Information specified for each connection was taken from the latest corresponding Frankel Steel Limited steel shop drawing for that connection

C6.0 INTERIOR COLUMN BRACING CAPACITY TABLES

C6.1 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 58 (NORTH – SOUTH)

Column 58 - North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] North Connection Conn. Type	North Connection T [kip]	North Connection C [kip]	South Connection Conn. Type	South Connection T [kip]	South Connection C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only) North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		90	90	1.8	Knife	14.5	147.0	Knife	8.1	84.0	98	155	98	5463%
Floor 47	134	322	232	6.4	-	0	0	Knife	8.1	84.0	84	8	8	126%
Floor 46		455	133	9.1	Header	73.3	320.1	Knife	8.1	84.0	157	320	157	1730%
Floor 45	637	579	124	11.6	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	832%
Floor 44		701	123	14.0	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	687%
Floor 43	989	825	124	16.5	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	584%
Floor 42		947	122	18.9	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	509%
Floor 41	1141	1071	124	21.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	450%
Floor 40		1194	123	23.9	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	404%
Floor 39	1393	1270	77	25.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	379%
Floor 38		1393	123	27.9	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	346%
Floor 37	1645	1518	124	30.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	318%
Floor 36		1641	123	32.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	294%
Floor 35	1897	1765	125	35.3	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	273%
Floor 34		1889	124	37.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	255%
Floor 33	2149	2014	125	40.3	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	239%
Floor 32		2139	124	42.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	225%
Floor 31	2401	2264	126	45.3	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	213%
Floor 30		2389	125	47.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	202%
Floor 29	2653	2516	128	50.3	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	192%
Floor 28		2641	125	52.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	182%
Floor 27	3905	2768	127	55.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	174%
Floor 26		2894	126	57.9	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	167%
Floor 25	3157	3021	127	60.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	160%
Floor 24		3147	126	62.9	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	153%
Floor 23	3409	3307	160	66.1	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	146%
Floor 22		3466	159	69.3	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	139%
Floor 21	3723	3627	161	72.5	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	133%
Floor 20		3758	131	75.2	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	128%
Floor 19	4006	3891	134	77.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	124%
Floor 18		4019	128	80.4	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	120%
Floor 17	4258	4148	129	83.0	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	116%
Floor 16		4277	129	85.5	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	113%
Floor 15	4510	4407	130	88.1	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	109%
Floor 14		4535	129	90.7	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	106%
Floor 13	4762	4666	130	93.3	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	103%
Floor 12		4795	130	95.9	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	100%
Floor 11	5014	4927	131	98.5	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	98%
Floor 10		5061	135	101.2	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	95%
Floor 9	5266	5192	131	103.8	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	93%
Floor 8		5323	130	106.5	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	91%
Floor 7	5518	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.2 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 58 (EAST – WEST)

Column 58 - East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		90	90	1.8	Header	16.4	122.5	Header	20.7	144.8	161	143	143	7943%
Floor 47	134	322	232	6.4	Header Pl	89.7	200.4	Header	25.0	167.0	257	NC	257	3884%
Floor 46		455	133	9.1	Header Pl	20.5	89.1	Header	29.3	189.3	210	NC	210	2307%
Floor 45	637	579	124	11.6	Header	12.2	100.2	Header	36.2	208.8	221	136	136	1170%
Floor 44		701	123	14.0	Header	12.2	100.2	Header	16.4	122.5	135	117	117	832%
Floor 43	989	825	124	16.5	Header	12.2	100.2	Header	36.2	208.8	221	136	136	827%
Floor 42		947	122	18.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	616%
Floor 41	1141	1071	124	21.4	Header	12.2	100.2	Header	36.2	208.8	221	136	136	637%
Floor 40		1194	123	23.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	489%
Floor 39	1393	1270	77	25.4	Header	12.2	100.2	Header	36.2	208.8	221	136	136	537%
Floor 38		1393	123	27.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	419%
Floor 37	1645	1518	124	30.4	Header	12.2	100.2	Header	36.2	208.8	221	136	136	450%
Floor 36		1641	123	32.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	356%
Floor 35	1897	1765	125	35.3	Header	12.2	100.2	Header	36.2	208.8	221	136	136	386%
Floor 34		1889	124	37.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	309%
Floor 33	2149	2014	125	40.3	Header	12.2	100.2	Header	36.2	208.8	221	136	136	339%
Floor 32		2139	124	42.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	273%
Floor 31	2401	2264	126	45.3	Header	12.2	100.2	Header	36.2	208.8	221	136	136	301%
Floor 30		2389	125	47.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	244%
Floor 29	2653	2515	126	50.3	Header	12.2	100.2	Header	36.2	208.8	221	136	136	271%
Floor 28		2641	125	52.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	221%
Floor 27	3905	2768	127	55.4	Header	12.2	100.2	Header	36.2	208.8	221	136	136	246%
Floor 26		2894	126	57.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	202%
Floor 25	3157	3021	127	60.4	Header	12.2	100.2	Header	36.2	208.8	221	136	136	226%
Floor 24		3147	126	62.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	185%
Floor 23	3409	3307	160	66.1	Header	12.2	100.2	Header	36.2	208.8	221	136	136	206%
Floor 22		3466	159	69.3	Header	12.2	100.2	Header	16.4	122.5	135	117	117	168%
Floor 21	3723	3627	161	72.5	Header	12.2	100.2	Header	36.2	208.8	221	136	136	188%
Floor 20		3758	131	75.2	Header	12.2	100.2	Header	16.4	122.5	135	117	117	155%
Floor 19	4006	3891	134	77.8	Header	12.2	100.2	Header	36.2	208.8	221	136	136	175%
Floor 18		4019	128	80.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	145%
Floor 17	4258	4148	129	83.0	Header	12.2	100.2	Header	36.2	208.8	221	136	136	164%
Floor 16		4277	129	85.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	136%
Floor 15	4510	4407	130	88.1	Header	12.2	100.2	Header	36.2	208.8	221	136	136	155%
Floor 14		4535	129	90.7	Header	12.2	100.2	Header	16.4	122.5	135	117	117	129%
Floor 13	4762	4666	130	93.3	Header	12.2	100.2	Header	36.2	208.8	221	136	136	140%
Floor 12		4795	130	95.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	122%
Floor 11	5014	4927	131	98.5	Header	12.2	100.2	Header	36.2	208.8	221	136	136	138%
Floor 10		5061	135	101.2	Header	12.2	100.2	Header	16.4	122.5	135	117	117	115%
Floor 9	5266	5192	131	103.8	Header	12.2	100.2	Header	36.2	208.8	221	136	136	131%
Floor 8		5323	130	106.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	110%
Floor 7	5518	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 6		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 5		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 4		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 3		-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.3 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 59 (NORTH – SOUTH)

Column 59 - North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req Bracing Capacity
Roof		94	94	1.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	3770%
Floor 47	73	149	54	3.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	2395%
Floor 46		205	57	4.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	1731%
Floor 45	283	262	57	5.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	1357%
Floor 44		318	56	6.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	1117%
Floor 43	420	375	56	7.5	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	949%
Floor 42		431	56	8.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	826%
Floor 41	556	487	56	9.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	730%
Floor 40		544	56	10.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	654%
Floor 39	692	601	57	12.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	592%
Floor 38		657	56	13.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	541%
Floor 37	828	714	57	14.3	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	498%
Floor 36		771	57	15.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	461%
Floor 35	964	828	57	16.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	430%
Floor 34		885	57	17.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	402%
Floor 33	1100	942	57	18.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	378%
Floor 32		999	57	20.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	359%
Floor 31	1236	1056	57	21.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	337%
Floor 30		1114	57	22.3	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	319%
Floor 29	1372	1171	58	23.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	304%
Floor 28		1229	58	24.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	289%
Floor 27	1508	1287	58	25.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	276%
Floor 26		1345	58	26.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	264%
Floor 25	1644	1403	59	28.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	253%
Floor 24		1462	58	29.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	243%
Floor 23	1865	1536	74	30.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	232%
Floor 22		1609	74	32.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	221%
Floor 21	1950	1684	74	33.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	211%
Floor 20		1743	59	34.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	204%
Floor 19	2103	1802	59	36.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	197%
Floor 18		1861	59	37.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	191%
Floor 17	2239	1921	60	38.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	185%
Floor 16		1980	60	39.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	180%
Floor 15	2375	2040	60	40.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	174%
Floor 14		2100	60	42.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	169%
Floor 13	2511	2160	60	43.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	165%
Floor 12		2220	60	44.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	160%
Floor 11	2647	2281	60	45.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	156%
Floor 10		2343	63	46.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	152%
Floor 9	2733	2404	61	48.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	148%
Floor 8		2465	61	49.3	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	144%
Floor 7	2919	–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 6		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 5		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 4		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 3		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 2		–	–	–	–	–	–	–	–	–	–	–	–	–

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

C6.4 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 59 (EAST – WEST)

Column 59 – East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		94	94	1.9	–	0	0	Header	13.8	144.8	145	14	14	734%
Floor 47	73	149	54	3.0	–	0	0	Header	11.0	122.5	122	11	11	370%
Floor 46		205	57	4.1	–	0	0	Header	11.0	122.5	122	11	11	267%
Floor 45	283	262	57	5.2	–	0	0	Header	11.0	122.5	122	11	11	209%
Floor 44		318	56	6.4	–	0	0	Header	11.0	122.5	122	11	11	173%
Floor 43	420	375	56	7.5	–	0	0	Header	No info	No info	No info	No info	No info	-
Floor 42		431	56	8.6	–	0	0	Header	11.0	122.5	122	11	11	127%
Floor 41	556	487	56	9.7	–	0	0	Header	No info	No info	No info	No info	No info	
Floor 40		544	56	10.9	–	0	0	Header	11.0	122.5	122	11	11	101%
Floor 39	692	601	57	12.0	–	0	0	Header	No info	No info	No info	No info	No info	–
Floor 38		657	56	13.1	–	0	0	Header	11.0	122.5	122	11	11	84%
Floor 37	828	714	57	14.3	–	0	0	Header	11.0	122.5	122	11	11	77%
Floor 36		771	57	15.4	–	0	0	Header	11.0	122.5	122	11	11	71%
Floor 35	964	828	57	16.6	–	0	0	Header	11.0	122.5	122	11	11	66%
Floor 34		885	57	17.7	–	0	0	Header	11.0	122.5	122	11	11	62%
Floor 33	1100	942	57	18.8	–	0	0	Header	11.0	122.5	122	11	11	58%
Floor 32		999	57	20.0	–	0	0	Header	11.0	122.5	122	11	11	55%
Floor 31	1236	1056	57	21.1	–	0	0	Header	11.0	122.5	122	11	11	52%
Floor 30		1114	57	22.3	–	0	0	Header	11.0	122.5	122	11	11	49%
Floor 29	1372	1171	58	23.4	–	0	0	Header	11.0	122.5	122	11	11	47%
Floor 28		1229	58	24.6	–	0	0	Header	11.0	122.5	122	11	11	45%
Floor 27	1508	1287	58	25.7	–	0	0	Header	11.0	122.5	122	11	11	43%
Floor 26		1345	58	26.9	–	0	0	Header	11.0	122.5	122	11	11	41%
Floor 25	1644	1403	59	28.1	–	0	0	Header	11.0	122.5	122	11	11	39%
Floor 24		1462	58	29.2	–	0	0	Header	11.0	122.5	122	11	11	38%
Floor 23	1865	1536	74	30.7	–	0	0	Header	11.0	122.5	122	11	11	36%
Floor 22		1609	74	32.2	–	0	0	Header	11.0	122.5	122	11	11	34%
Floor 21	1950	1684	74	33.7	–	0	0	Header	11.0	122.5	122	11	11	33%
Floor 20		1743	59	34.9	–	0	0	Header	11.0	122.5	122	11	11	32%
Floor 19	2103	1802	59	36.0	–	0	0	Header	11.0	122.5	122	11	11	30%
Floor 18		1861	59	37.2	–	0	0	Header	11.0	122.5	122	11	11	30%
Floor 17	2239	1921	60	38.4	–	0	0	Header	11.0	122.5	122	11	11	29%
Floor 16		1980	60	39.6	–	0	0	Header	11.0	122.5	122	11	11	28%
Floor 15	2375	2040	60	40.8	–	0	0	Header	11.0	122.5	122	11	11	27%
Floor 14		2100	60	42.0	–	0	0	Header	11.0	122.5	122	11	11	26%
Floor 13	2511	2160	60	43.2	–	0	0	Header	11.0	122.5	122	11	11	25%
Floor 12		2220	60	44.4	–	0	0	Header	11.0	122.5	122	11	11	25%
Floor 11	2647	2281	60	45.6	–	0	0	Header	11.0	122.5	122	11	11	24%
Floor 10		2343	63	46.9	–	0	0	Header	11.0	122.5	122	11	11	23%
Floor 9	2733	2404	61	48.1	–	0	0	Header	11.0	122.5	122	11	11	23%
Floor 8		2465	61	49.3	–	0	0	Header	11.0	122.5	122	11	11	22%
Floor 7	2919	–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 6		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 5		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 4		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 3		–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 2		–	–	–	–	–	–	–	–	–	–	–	–	–

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

C6.5 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 60 (NORTH – SOUTH)

Column 60 - North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		23	23	0.5	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	15210%
Floor 47	77	75	52	1.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	4732%
Floor 46		126	51	2.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	2819%
Floor 45	311	185	59	3.7	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	1910%
Floor 44		245	59	4.9	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	1453%
Floor 43	462	304	59	6.1	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	1170%
Floor 42		363	59	7.3	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	979%
Floor 41	612	423	59	8.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	841%
Floor 40		482	60	9.6	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	737%
Floor 39	762	542	60	10.8	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	656%
Floor 38		602	60	12.0	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	590%
Floor 37	912	662	60	13.2	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	537%
Floor 36		723	61	14.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	492%
Floor 35	1062	784	61	15.7	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	454%
Floor 34		845	61	16.9	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	421%
Floor 33	1212	905	61	18.1	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	393%
Floor 32		966	61	19.3	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	368%
Floor 31	1362	1028	61	20.6	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	346%
Floor 30		1089	62	21.8	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	327%
Floor 29	1512	1151	62	23.0	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	309%
Floor 28		1213	62	24.3	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	293%
Floor 27	1662	1275	62	25.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	279%
Floor 26		1337	62	26.7	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	266%
Floor 25	1812	1398	62	28.0	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	254%
Floor 24		1461	62	29.2	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	243%
Floor 23	1962	1540	79	30.8	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	231%
Floor 22		1619	80	32.4	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	220%
Floor 21	2150	1699	80	34.0	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	209%
Floor 20		1763	64	35.3	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	202%
Floor 19	2319	1827	64	36.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	195%
Floor 18		1891	64	37.8	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	188%
Floor 17	2469	1954	64	39.1	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	182%
Floor 16		2018	64	40.4	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	176%
Floor 15	2619	2083	64	41.7	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	171%
Floor 14		2147	65	42.9	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	166%
Floor 13	2769	2212	65	44.2	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	161%
Floor 12		2277	65	45.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	156%
Floor 11	2919	2342	65	46.8	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	152%
Floor 10		2408	67	48.2	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	148%
Floor 9	3069	2473	65	49.5	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	144%
Floor 8		2539	66	50.8	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	140%
Floor 7	3234	2678	139	53.6	Knife	6.0	63.0	Knife	12.4	126.0	132	75	75	141%
Floor 6		2721	43	54.4	Knife	8.1	84.0	Knife	8.1	84.0	92	92	92	169%
Floor 5	3487	2883	162	57.7	Knife	10.3	105.0	Knife	11.2	168.0	178	116	116	201%
Floor 4		2988	105	59.8	Knife	8.1	84.0	Knife	10.3	106.0	113	94	94	158%
Floor 3	3850	3174	186	63.5	Knife	14.9	147.0	Knife	19.1	189.0	204	166	166	262%
Floor 2		3257	83	65.1	Knife	8.5	84.0	Knife	12.7	126.0	134	97	97	148%

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

C6.6 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 60 (EAST – WEST)

Column 60 - East-West

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		23	23	0.5	Header	17.6	125.3	Header	10.7	122.5	140	138	138	29071%
Floor 47	77	75	52	1.5	–	0	0	Header	10.7	122.5	122	11	11	711%
Floor 46		126	51	2.5	–	0	0	Header	16.3	181.9	182	16	16	644%
Floor 45	311	185	59	3.7	–	0	0	Header	10.7	129.9	130	11	11	288%
Floor 44		245	59	4.9	–	0	0	Header	10.7	129.9	130	11	11	218%
Floor 43	462	304	59	6.1	–	0	0	Header	10.7	122.5	122	11	11	176%
Floor 42		363	59	7.3	–	0	0	Header	10.7	122.5	122	11	11	147%
Floor 41	612	423	59	8.5	–	0	0	Header	10.7	122.5	122	11	11	126%
Floor 40		482	60	9.6	–	0	0	Header	10.7	122.5	122	11	11	111%
Floor 39	762	542	60	10.8	–	0	0	Header	10.7	122.5	122	11	11	99%
Floor 38		602	60	12.0	–	0	0	Header	10.7	122.5	122	11	11	89%
Floor 37	912	662	60	13.2	–	0	0	Header	10.7	122.5	122	11	11	81%
Floor 36		723	61	14.5	–	0	0	Header	10.7	122.5	122	11	11	74%
Floor 35	1062	784	61	15.7	–	0	0	Header	10.7	122.5	122	11	11	68%
Floor 34		845	61	16.9	–	0	0	Header	10.7	122.5	122	11	11	63%
Floor 33	1212	905	61	18.1	–	0	0	Header	10.7	122.5	122	11	11	59%
Floor 32		966	61	19.3	–	0	0	Header	10.7	122.5	122	11	11	56%
Floor 31	1362	1028	61	20.6	–	0	0	Header	10.7	122.5	122	11	11	52%
Floor 30		1089	62	21.8	–	0	0	Header	10.7	122.5	122	11	11	49%
Floor 29	1512	1151	62	23.0	–	0	0	Header	10.7	122.5	122	11	11	46%
Floor 28		1213	62	24.3	–	0	0	Header	10.7	122.5	122	11	11	44%
Floor 27	1662	1275	62	25.5	–	0	0	Header	10.7	122.5	122	11	11	42%
Floor 26		1337	62	26.7	–	0	0	Header	10.7	122.5	122	11	11	40%
Floor 25	1812	1398	62	28.0	–	0	0	Header	10.7	122.5	122	11	11	38%
Floor 24		1461	62	29.2	–	0	0	Header	10.7	122.5	122	11	11	37%
Floor 23	1962	1540	79	30.8	–	0	0	Header	10.7	122.5	122	11	11	35%
Floor 22		1619	80	32.4	–	0	0	Header	10.7	122.5	122	11	11	33%
Floor 21	2150	1699	80	34.0	–	0	0	Header	10.7	122.5	122	11	11	31%
Floor 20		1763	64	35.3	–	0	0	Header	10.7	122.5	122	11	11	30%
Floor 19	2319	1827	64	36.5	–	0	0	Header	10.7	122.5	122	11	11	29%
Floor 18		1891	64	37.8	–	0	0	Header	10.7	122.5	122	11	11	28%
Floor 17	2469	1954	64	39.1	–	0	0	Header	10.7	122.5	122	11	11	27%
Floor 16		2018	64	40.4	–	0	0	Header	10.7	122.5	122	11	11	26%
Floor 15	2619	2083	64	41.7	–	0	0	Header	10.7	122.5	122	11	11	26%
Floor 14		2147	65	42.9	–	0	0	Header	10.7	122.5	122	11	11	25%
Floor 13	2769	2212	65	44.2	–	0	0	Header	10.7	122.5	122	11	11	24%
Floor 12		2277	65	45.5	–	0	0	Header	10.7	122.5	122	11	11	23%
Floor 11	2919	2342	65	46.8	–	0	0	Header	10.7	122.5	122	11	11	23%
Floor 10		2408	67	48.2	–	0	0	Header	10.7	122.5	122	11	11	22%
Floor 9	3069	2473	65	49.5	–	0	0	Header	10.7	122.5	122	11	11	22%
Floor 8		2539	66	50.8	–	0	0	Header	10.7	122.5	122	11	11	21%
Floor 7	3234	2678	139	53.6	–	0	0	Header	11.6	129.9	130	12	12	22%
Floor 6		2721	43	54.4	Header	7.5	100.2	Header	12.0	143.8	151	112	112	206%
Floor 5	3487	2883	162	57.7	–	0	0	Header	69.5	350.8	351	70	70	121%
Floor 4		2988	105	59.8	–	0	0	Header	No Info	No Info	No info	No info	No info	–
Floor 3	3850	3174	186	63.5	–	0	0	Header	40.4	292.3	292	40	40	64%
Floor 2		3257	83	65.1	–	0	0	Header	25.0	167.0	167	25	25	39%

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

C6.7 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 61 (NORTH – SOUTH)

Column 61 – North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	P [kip]	Conn. Type	T [kip]	P [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		213	213	4.3	Knife	8.1	84.0	Knife	8.1	84.0	92	92	92	2162%
Floor 47	138	353	140	7.1	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	1008%
Floor 46		459	106	9.2	Seated	12.4	126.0	Knife	8.1	84.0	96	134	96	1050%
Floor 45	605	543	84	10.9	Seated	42	42	Knife	8.1	84.0	126	50	50	461%
Floor 44		626	83	12.5	Seated	42	42	Knife	8.1	84.0	126	50	50	400%
Floor 43	810	710	83	14.2	Seated	42	42	Knife	8.1	84.0	126	50	50	353%
Floor 42		792	83	15.8	Seated	42	42	Knife	8.1	84.0	126	50	50	316%
Floor 41	1014	875	83	17.5	Seated	42	42	Knife	8.1	84.0	126	50	50	286%
Floor 40		958	82	19.2	Seated	42	42	Knife	8.1	84.0	126	50	50	262%
Floor 39	1218	1041	83	20.8	Seated	42	42	Knife	8.1	84.0	126	50	50	241%
Floor 38		1123	82	22.5	Seated	42	42	Knife	8.1	84.0	126	50	50	223%
Floor 37	1422	1206	83	24.1	Seated	42	42	Knife	8.1	84.0	126	50	50	208%
Floor 36		1288	82	25.8	Seated	42	42	Knife	8.1	84.0	126	50	50	195%
Floor 35	1626	1371	83	27.4	Seated	42	42	Knife	8.1	84.0	126	50	50	183%
Floor 34		1454	83	29.1	Seated	42	42	Knife	8.1	84.0	126	50	50	172%
Floor 33	1830	1538	84	30.8	Seated	42	42	Knife	8.1	84.0	126	50	50	163%
Floor 32		1621	83	32.4	Seated	42	42	Knife	8.1	84.0	126	50	50	155%
Floor 31	2034	1705	84	34.1	Seated	42	42	Knife	8.1	84.0	126	50	50	147%
Floor 30		1788	83	35.8	Seated	42	42	Knife	8.1	84.0	126	50	50	140%
Floor 29	2258	1872	84	37.4	Seated	42	42	Knife	8.1	84.0	126	50	50	134%
Floor 28		1956	84	39.1	Seated	42	42	Knife	8.1	84.0	126	50	50	128%
Floor 27	2442	2040	84	40.8	Seated	42	42	Knife	8.1	84.0	126	50	50	123%
Floor 26		2124	84	42.5	Seated	42	42	Knife	8.1	84.0	126	50	50	118%
Floor 25	2646	2208	84	44.2	Seated	42	42	Knife	8.1	84.0	126	50	50	114%
Floor 24		2292	84	45.8	Seated	42	42	Knife	8.1	84.0	126	50	50	109%
Floor 23	2850	2397	105	47.9	Seated	42	42	Knife	8.1	84.0	126	50	50	105%
Floor 22		2501	105	50.0	Seated	42	42	Knife	8.1	84.0	126	50	50	100%
Floor 21	3094	2607	105	52.1	Seated	42	42	Knife	8.1	84.0	126	50	50	96%
Floor 20		2690	84	53.8	Seated	42	42	Knife	8.1	84.0	126	50	50	93%
Floor 19	3318	2774	84	55.5	Seated	42	42	Knife	8.1	84.0	126	50	50	90%
Floor 18		2859	85	57.2	Seated	42	42	Knife	8.1	84.0	126	50	50	88%
Floor 17	3522	2945	86	58.9	Seated	42	42	Knife	8.1	84.0	126	50	50	85%
Floor 16		3030	85	60.6	Seated	42	42	Knife	8.1	84.0	126	50	50	83%
Floor 15	3726	3116	86	62.3	Seated	42	42	Knife	8.1	84.0	126	50	50	80%
Floor 14		3201	85	64.0	Seated	42	42	Knife	8.1	84.0	126	50	50	78%
Floor 13	3930	3288	86	65.8	Seated	42	42	Knife	8.1	84.0	126	50	50	76%
Floor 12		3373	85	67.5	Seated	42	42	Knife	8.1	84.0	126	50	50	74%
Floor 11	4134	3459	86	69.2	Seated	42	42	Knife	8.1	84.0	126	50	50	72%
Floor 10		3543	84	70.9	Seated	42	42	Knife	8.1	84.0	126	50	50	71%
Floor 9	4338	3629	86	72.6	Seated	42	42	Knife	8.1	84.0	126	50	50	69%
Floor 8		3715	85	74.3	Knife	42	42	Knife	8.1	84.0	126	50	50	67%
Floor 7	4247	2349	-1366	47.0	Knife	59.0	692.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		1001	-1349	20.0	Knife	8.1	84.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 5	4475	959	-41	19.2	Knife	8.1	84.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		1084	124	21.7	-	10.3	105.0	Knife	6.0	63.0	73	111	73	338%
Floor 3	190	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.8 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 61 (EAST – WEST)

Column 61 – East-West

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]: West Connection Conn. Type	West Connection T [kip]	West Connection P [kip]	East Connection Conn. Type	East Connection T [kip]	East Connection P [kip]	Total Bracing Capacity in the East-West Direction (Provided by the Connections Only): West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		213	213	4.3	Header	20.7	144.8	Header	No info	No info	No info	No info	No info	-
Floor 47	138	353	140	7.1	Header	25.7	170.7	Header	43.5	241.3	267	214	214	3034%
Floor 46		459	106	9.2	Header	37.3	213.4	Header	88.0	322.9	360	301	301	3205%
Floor 45	605	543	84	10.9	Header	36.2	208.8	Header	16.4	122.5	159	225	159	1461%
Floor 44		626	83	12.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1109%
Floor 43	810	710	83	14.2	Header	36.2	208.8	Header	16.4	122.5	159	225	159	1118%
Floor 42		792	83	15.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	877%
Floor 41	1014	875	83	17.5	Header	36.2	208.8	Header	16.4	122.5	159	225	159	908%
Floor 40		958	82	19.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	725%
Floor 39	1218	1041	83	20.8	Header	36.2	208.8	Header	16.4	122.5	159	225	159	762%
Floor 38		1123	82	22.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	619%
Floor 37	1422	1206	83	24.1	Header	36.2	208.8	Header	16.4	122.5	159	225	159	658%
Floor 36		1288	82	25.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	539%
Floor 35	1626	1371	83	27.4	Header	36.2	208.8	Header	16.4	122.5	159	225	159	579%
Floor 34		1454	83	29.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	478%
Floor 33	1830	1538	84	30.8	Header	36.2	208.8	Header	16.4	122.5	159	225	159	516%
Floor 32		1621	83	32.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	429%
Floor 31	2034	1705	84	34.1	Header	36.2	208.8	Header	16.4	122.5	159	225	159	465%
Floor 30		1788	83	35.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	388%
Floor 29	2258	1872	84	37.4	Header	36.2	208.8	Header	16.4	122.5	159	225	159	424%
Floor 28		1956	84	39.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	355%
Floor 27	2442	2040	84	40.8	Header	36.2	208.8	Header	16.4	122.5	159	225	159	389%
Floor 26		2124	84	42.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	327%
Floor 25	2646	2208	84	44.2	Header	36.2	208.8	Header	16.4	122.5	159	225	159	359%
Floor 24		2292	84	45.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	303%
Floor 23	2850	2397	105	47.9	Header	36.2	208.8	Header	16.4	122.5	159	225	159	331%
Floor 22		2501	105	50.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	278%
Floor 21	3094	2607	105	52.1	Header	36.2	208.8	Header	16.4	122.5	159	225	159	304%
Floor 20		2690	84	53.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	258%
Floor 19	3318	2774	84	55.5	Header	36.2	208.8	Header	16.4	122.5	159	225	159	286%
Floor 18		2859	85	57.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	243%
Floor 17	3522	2945	86	58.9	Header	36.2	208.8	Header	16.4	122.5	159	225	159	269%
Floor 16		3030	85	60.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	229%
Floor 15	3726	3116	86	62.3	Header	36.2	208.8	Header	16.4	122.5	159	225	159	255%
Floor 14		3201	85	64.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	217%
Floor 13	3930	3288	86	65.8	Header	36.2	208.8	Header	16.4	122.5	159	225	159	241%
Floor 12		3373	85	67.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	206%
Floor 11	4134	3459	86	69.2	Header	36.2	208.8	Header	16.4	122.5	159	225	159	229%
Floor 10		3543	84	70.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	196%
Floor 9	4338	3629	86	72.6	Header	36.2	208.8	Header	16.4	122.5	159	225	159	219%
Floor 8		3715	85	74.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	187%
Floor 7	4247	2349	1366	47.0	Header	26.5	174.5	Header	33.6	211.6	238	208	208	443%
Floor 6		1001	-1349	20.0	Header	30.0	199.5	Header	25.7	170.7	201	225	201	1003%
Floor 5	4475	959	-41	19.2	Header	48.6	283.0	Header	26.5	174.5	223	309	223	1163%
Floor 4		1084	124	21.7	Header	33.6	211.6	Header	29.3	189.3	223	241	223	1028%
Floor 3	190	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

JA-4159

C6.9 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 62 (NORTH – SOUTH)

Column 62 - North-South															
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kips]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	P [kip]	Conn. Type	T [kip]	P [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		239	239	4.8	Knife	8.1	84.0	Header	83.8	431.5	440	168	168	3504%
Floor 47	135	208	-32	4.2	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	1762%
Floor 46		256	48	5.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	1392%
Floor 45	312	299	43	6.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	1190%
Floor 44		342	43	6.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	1039%
Floor 43	441	385	43	7.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	923%
Floor 42		428	43	8.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	831%
Floor 41	567	471	43	9.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	756%
Floor 40		513	43	10.3	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	693%
Floor 39	693	556	43	11.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	640%
Floor 38		599	43	12.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	594%
Floor 37	819	641	43	12.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	555%
Floor 36		684	43	13.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	520%
Floor 35	945	726	43	14.5	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	490%
Floor 34		769	43	15.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	462%
Floor 33	1071	812	43	16.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	438%
Floor 32		855	43	17.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	419%
Floor 31	1197	898	43	18.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	396%
Floor 30		941	43	18.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	378%
Floor 29	1323	985	43	19.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	361%
Floor 28		1028	43	20.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	346%
Floor 27	1449	1071	43	21.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	332%
Floor 26		1115	44	22.3	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	319%
Floor 25	1575	1159	44	23.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	307%
Floor 24		1203	44	24.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	296%
Floor 23	1701	1299	97	26.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	274%
Floor 22		1354	55	27.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	263%
Floor 21	1855	1409	55	28.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	252%
Floor 20		1452	44	29.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	245%
Floor 19	1995	1497	44	29.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	238%
Floor 18		1541	45	30.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	231%
Floor 17	2121	1586	45	31.7	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	224%
Floor 16		1631	45	32.6	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	218%
Floor 15	2247	1676	45	33.5	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	212%
Floor 14		1721	45	34.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	207%
Floor 13	2373	1766	45	35.3	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	201%
Floor 12		1811	45	36.2	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	196%
Floor 11	2499	1856	45	37.1	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	192%
Floor 10		1901	45	38.0	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	187%
Floor 9	2625	1946	45	38.9	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	183%
Floor 8		1991	45	39.8	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	179%
Floor 7	2766	2002	11	40.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	172%
Floor 6		1393	-610	27.9	Truss Cnx	NC	NC	Knife	10.3	105.0	NC	NC	NC	>100
Floor 5	2940	1058	-335	21.2	Truss Cnx	NC	NC	Knife	6.0	63.0	NC	NC	NC	>100
Floor 4		1130	72	22.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	305%
Floor 3	1381	1181	51	23.6	Knife	20.0	84.0	Knife	6.0	63.0	83	90	83	351%
Floor 2		1235	53	24.7	Knife	8.5	84.0	Knife	10.6	105.0	113	95	95	363%

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

C6.10 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 62 (EAST – WEST)

Column 62 – East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kips]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	P [kip]	Conn. Type	T [kip]	P [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		239	239	4.8	Header	20.7	144.8	Header	54.9	292.3	313	200	200	4168%
Floor 47	135	208	-32	4.2	Header	16.4	122.5	Knife	10.3	105.0	121	133	121	2922%
Floor 46		256	48	5.1	Header	12.2	100.2	Knife	8.1	84.0	96	108	96	1892%
Floor 45	312	299	43	6.0	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	1680%
Floor 44		342	43	6.8	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	1467%
Floor 43	441	385	43	7.7	Header	No Info	No Info	Knife	8.1	84.0	No info	No info	No info	–
Floor 42		428	43	8.6	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	1174%
Floor 41	567	471	43	9.4	Header	No Info	No Info	Knife	8.1	84.0	No info	No info	No info	–
Floor 40		513	43	10.3	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	978%
Floor 39	693	556	43	11.1	Header	No Info	No Info	Knife	8.1	84.0	No info	No info	No info	–
Floor 38		599	43	12.0	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	839%
Floor 37	819	641	43	12.8	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	783%
Floor 36		684	43	13.7	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	734%
Floor 35	945	726	43	14.5	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	691%
Floor 34		769	43	15.4	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	653%
Floor 33	1071	812	43	16.2	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	619%
Floor 32		855	43	17.1	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	587%
Floor 31	1197	898	43	18.0	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	559%
Floor 30		941	43	18.8	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	534%
Floor 29	1323	985	43	19.7	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	510%
Floor 28		1028	43	20.6	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	489%
Floor 27	1449	1071	43	21.4	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	469%
Floor 26		1115	44	22.3	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	450%
Floor 25	1575	1159	44	23.2	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	433%
Floor 24		1203	44	24.1	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	418%
Floor 23	1701	1299	97	26.0	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	387%
Floor 22		1354	55	27.1	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	371%
Floor 21	1855	1409	55	28.2	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	357%
Floor 20		1452	44	29.0	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	346%
Floor 19	1995	1497	44	29.9	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	336%
Floor 18		1541	45	30.8	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	326%
Floor 17	2121	1586	45	31.7	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	317%
Floor 16		1631	45	32.6	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	308%
Floor 15	2247	1676	45	33.5	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	300%
Floor 14		1721	45	34.4	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	292%
Floor 13	2373	1766	45	35.3	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	284%
Floor 12		1811	45	36.2	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	277%
Floor 11	2499	1856	45	37.1	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	271%
Floor 10		1901	45	38.0	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	264%
Floor 9	2625	1946	45	38.9	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	258%
Floor 8		1991	45	39.8	Header	16.4	122.5	Knife	8.1	84.0	100	131	100	252%
Floor 7	2766	2002	11	40.0	Header	16.4	122.5	Knife (offset)	11.7	126.0	142	134	134	335%
Floor 6		1393	-610	27.9	Header	20.7	144.8	Knife (offset)	8.1	84.0	105	153	105	376%
Floor 5	2940	1058	-335	21.2	Header	89.8	350.8	Knife (offset)	8.1	84.0	174	359	174	822%
Floor 4		1130	72	22.6	Header	7.9	77.9	Knife (offset)	8.1	84.0	92	86	86	381%
Floor 3	1381	1181	51	23.6	Header	12.2	100.2	Fin (offset)	103.8	103.8	116	204	116	491%
Floor 2		1235	53	24.7	Header	16.4	122.5	Fin (offset)	103.8	103.8	120	226	120	487%

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

C6.11 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 63 (NORTH – SOUTH)

Column 63 - North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		21	21	0.4	Knife	8.1	84.0	Header	12.2	100.2	108	96	96	23364%
Floor 47	98	63	43	1.3	M-Conn	NC	NC	Header	13.6	107.8	NC	NC	NC	>100
Floor 46		100	37	2.0	Knife	6.0	63.0	Header	25.0	167.0	173	88	88	4380%
Floor 45	304	157	56	3.1	Knife	6.0	63.0	Seated	42	42	48	105	48	1533%
Floor 44		213	56	4.3	Knife	6.0	63.0	Seated	42	42	48	105	48	1129%
Floor 43	441	269	56	5.4	Knife	6.0	63.0	Seated	42	42	48	105	48	893%
Floor 42		325	56	6.5	Knife	6.0	63.0	Seated	42	42	48	105	48	739%
Floor 41	577	381	56	7.6	Knife	6.0	63.0	Seated	42	42	48	105	48	630%
Floor 40		438	56	8.8	Knife	6.0	63.0	Seated	42	42	48	105	48	549%
Floor 39	713	494	56	9.9	Knife	6.0	63.0	Seated	42	42	48	105	48	486%
Floor 38		551	57	11.0	Knife	6.0	63.0	Seated	42	42	48	105	48	438%
Floor 37	849	608	57	12.2	Knife	6.0	63.0	Seated	42	42	48	105	48	395%
Floor 36		665	57	13.3	Knife	6.0	63.0	Seated	42	42	48	105	48	361%
Floor 35	985	722	57	14.4	Knife	6.0	63.0	Seated	42	42	48	105	48	333%
Floor 34		779	57	15.6	Knife	6.0	63.0	Seated	42	42	48	105	48	308%
Floor 33	1121	837	58	16.7	Knife	6.0	63.0	Seated	42	42	48	105	48	287%
Floor 32		894	58	17.9	Knife	6.0	63.0	Seated	42	42	48	105	48	268%
Floor 31	1257	952	58	19.0	Knife	6.0	63.0	Seated	42	42	48	105	48	252%
Floor 30		1010	58	20.2	Knife	6.0	63.0	Seated	42	42	48	105	48	238%
Floor 29	1393	1069	58	21.4	Knife	6.0	63.0	Seated	42	42	48	105	48	225%
Floor 28		1127	58	22.5	Knife	6.0	63.0	Seated	42	42	48	105	48	213%
Floor 27	1529	1186	59	23.7	Knife	6.0	63.0	Seated	42	42	48	105	48	202%
Floor 26		1244	59	24.9	Knife	6.0	63.0	Seated	42	42	48	105	48	193%
Floor 25	1665	1302	58	26.0	Knife	6.0	63.0	Seated	42	42	48	105	48	184%
Floor 24		1361	58	27.2	Knife	6.0	63.0	Seated	42	42	48	105	48	176%
Floor 23	1801	1434	73	28.7	Knife	6.0	63.0	Seated	42	42	48	105	48	167%
Floor 22		1507	73	30.1	Knife	6.0	63.0	Seated	42	42	48	105	48	159%
Floor 21	1969	1581	74	31.6	Knife	6.0	63.0	Seated	42	42	48	105	48	152%
Floor 20		1640	59	32.8	Knife	6.0	63.0	Seated	42	42	48	105	48	146%
Floor 19	2121	1699	59	34.0	Knife	6.0	63.0	Seated	42	42	48	105	48	141%
Floor 18		1759	60	35.2	Knife	6.0	63.0	Seated	42	42	48	105	48	136%
Floor 17	2257	1819	60	36.4	Knife	6.0	63.0	Seated	42	42	48	105	48	132%
Floor 16		1879	60	37.6	Knife	6.0	63.0	Seated	42	42	48	105	48	128%
Floor 15	2393	1939	60	38.8	Knife	6.0	63.0	Seated	42	42	48	105	48	124%
Floor 14		1999	60	40.0	Knife	6.0	63.0	Seated	42	42	48	105	48	120%
Floor 13	2529	2059	60	41.2	Knife	6.0	63.0	Seated	42	42	48	105	48	117%
Floor 12		2119	60	42.4	Knife	6.0	63.0	Seated	42	42	48	105	48	113%
Floor 11	2665	2180	60	43.6	Knife	6.0	63.0	Seated	42	42	48	105	48	110%
Floor 10		2235	56	44.7	Knife	6.0	63.0	Seated	42	42	48	105	48	107%
Floor 9	2801	2296	61	45.9	Knife	6.0	63.0	Seated	42	42	48	105	48	105%
Floor 8		2357	61	47.1	Knife	6.0	63.0	Seated	42	42	48	105	48	102%
Floor 7	2962	2478	121	49.6	Knife	6.0	63.0	Knife	8.1	84.0	90	71	71	144%
Floor 6		2532	53	50.6	Fin	103.8	103.8	Fin	103.8	103.8	208	208	208	410%
Floor 5	3242	2703	171	54.1	Knife	6.0	63.0	Knife	10.3	105.0	111	73	73	136%
Floor 4		2752	49	55.0	Knife	6.0	63.0	-	0	0	6	63	6	11%
Floor 3	3545	2865	114	57.3	Knife	6.4	63.0	Knife	8.5	63.0	69	71	69	121%
Floor 2		2921	57	58.5	Knife	6.4	63.0	–	0	0	6	63	6	11%

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

C6.12 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 63 (EAST – WEST)

Column 63 – East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		21	21	0.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	33768%
Floor 47	98	63	43	1.3	Header	16.4	122.5	Header	25.0	167.0	183	148	148	11633%
Floor 46		100	37	2.0	Header	78.0	245.9	Header	17.2	126.2	205	263	205	10203%
Floor 45	304	157	56	3.1	Header	16.4	122.5	Header	12.2	115.1	132	135	132	4200%
Floor 44		213	56	4.3	Header	16.4	122.5	Header	12.2	115.1	132	135	132	3093%
Floor 43	441	269	56	5.4	Header	16.4	122.5	Header	12.2	115.1	132	135	132	2447%
Floor 42		325	56	6.5	Header	16.4	122.5	Header	12.2	115.1	132	135	132	2024%
Floor 41	577	381	56	7.6	Header	16.4	122.5	Header	12.2	115.1	132	135	132	1725%
Floor 40		438	56	8.8	Header	16.4	122.5	Header	12.2	115.1	132	135	132	1503%
Floor 39	713	494	56	9.9	Header	16.4	122.5	Header	12.2	115.1	132	135	132	1331%
Floor 38		551	57	11.0	Header	16.4	122.5	Header	12.2	115.1	132	135	132	1194%
Floor 37	849	608	57	12.2	Header	16.4	122.5	Header	12.2	115.1	132	135	132	1082%
Floor 36		665	57	13.3	Header	16.4	122.5	Header	12.2	115.1	132	135	132	989%
Floor 35	985	722	57	14.4	Header	16.4	122.5	Header	12.2	115.1	132	135	132	911%
Floor 34		779	57	15.6	Header	16.4	122.5	Header	12.2	115.1	132	135	132	844%
Floor 33	1121	837	58	16.7	Header	16.4	122.5	Header	12.2	115.1	132	135	132	786%
Floor 32		894	58	17.9	Header	16.4	122.5	Header	12.2	115.1	132	135	132	735%
Floor 31	1257	952	58	19.0	Header	16.4	122.5	Header	12.2	115.1	132	135	132	690%
Floor 30		1010	58	20.2	Header	16.4	122.5	Header	12.2	115.1	132	135	132	651%
Floor 29	1393	1069	58	21.4	Header	16.4	122.5	Header	12.2	115.1	132	135	132	615%
Floor 28		1127	58	22.5	Header	16.4	122.5	Header	12.2	115.1	132	135	132	583%
Floor 27	1529	1186	59	23.7	Header	16.4	122.5	Header	12.2	115.1	132	135	132	555%
Floor 26		1244	59	24.9	Header	16.4	122.5	Header	12.2	115.1	132	135	132	529%
Floor 25	1665	1302	58	26.0	Header	16.4	122.5	Header	12.2	115.1	132	135	132	505%
Floor 24		1361	58	27.2	Header	16.4	122.5	Header	12.2	115.1	132	135	132	483%
Floor 23	1801	1434	73	28.7	Header	16.4	122.5	Header	16.4	137.3	154	139	139	484%
Floor 22		1507	73	30.1	Header	16.4	122.5	Header	16.4	137.3	154	139	139	461%
Floor 21	1969	1581	74	31.6	Header	16.4	122.5	Header	16.4	137.3	154	139	139	439%
Floor 20		1640	59	32.8	Header	16.4	122.5	Header	12.2	115.1	132	135	132	401%
Floor 19	2121	1699	59	34.0	Header	16.4	122.5	Header	12.2	115.1	132	135	132	387%
Floor 18		1759	60	35.2	Header	16.4	122.5	Header	12.2	115.1	132	135	132	374%
Floor 17	2257	1819	60	36.4	Header	16.4	122.5	Header	12.2	115.1	132	135	132	362%
Floor 16		1879	60	37.6	Header	16.4	122.5	Header	12.2	115.1	132	135	132	350%
Floor 15	2393	1939	60	38.8	Header	16.4	122.5	Header	12.2	115.1	132	135	132	339%
Floor 14		1999	60	40.0	Header	16.4	122.5	Header	12.2	115.1	132	135	132	329%
Floor 13	2529	2059	60	41.2	Header	16.4	122.5	Header	12.2	115.1	132	135	132	319%
Floor 12		2119	60	42.4	Header	16.4	122.5	Header	12.2	115.1	132	135	132	310%
Floor 11	2665	2180	60	43.6	Header	16.4	122.5	Header	12.2	115.1	132	135	132	302%
Floor 10		2235	56	44.7	Header	16.4	122.5	Header	12.2	115.1	132	135	132	294%
Floor 9	2801	2296	61	45.9	Header	16.4	122.5	Header	12.2	115.1	132	135	132	286%
Floor 8		2357	61	47.1	Header	16.4	122.5	Header	12.2	115.1	132	135	132	279%
Floor 7	2962	2478	121	49.6	Header	16.4	129.9	Header	16.4	122.5	139	146	139	280%
Floor 6		2532	53	50.6	Header	17.6	143.8	Header	12.2	100.2	118	156	118	233%
Floor 5	3242	2703	171	54.1	Header	89.8	350.8	Header	39.3	241.3	331	390	331	613%
Floor 4		2752	49	55.0	Header	12.2	100.2	Header	16.4	122.5	135	117	117	212%
Floor 3	3545	2865	114	57.3	Header	35.0	233.8	Header	20.7	144.8	180	255	180	314%
Floor 2		2923	57	58.5	–	0	0	Header	12.2	100.2	100	12	12	21%

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

C6.13 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 64 (NORTH – SOUTH)

Column 64 – North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		184	184	3.7	–	0	0	Knife	10.3	105.0	105	10	10	279%
Floor 47	152	254	70	5.1	–	0	0	Knife	14.0	210.0	210	14	14	277%
Floor 46		389	135	7.0	–	0	0	Knife	6.0	63.0	63	6	6	77%
Floor 45	537	472	84	9.4	–	0	0	Knife	6.0	63.0	63	6	6	64%
Floor 44		555	82	11.1	–	0	0	Knife	6.0	63.0	63	6	6	54%
Floor 43	722	638	83	12.8	–	0	0	Knife	6.0	63.0	63	6	6	47%
Floor 42		719	82	14.4	–	0	0	Knife	6.0	63.0	63	6	6	42%
Floor 41	906	801	82	16.0	–	0	0	Knife	6.0	63.0	63	6	6	37%
Floor 40		883	81	17.7	–	0	0	Knife	6.0	63.0	63	6	6	34%
Floor 39	1090	965	82	19.3	–	0	0	Knife	6.0	63.0	63	6	6	31%
Floor 38		1046	81	20.9	–	0	0	Knife	6.0	63.0	63	6	6	29%
Floor 37	1274	1128	82	22.6	–	0	0	Knife	6.0	63.0	63	6	6	27%
Floor 36		1210	82	24.2	–	0	0	Knife	6.0	63.0	63	6	6	26%
Floor 35	1458	1292	82	25.8	–	0	0	Knife	6.0	63.0	63	6	6	23%
Floor 34		1374	82	27.5	–	0	0	Knife	6.0	63.0	63	6	6	22%
Floor 33	1642	1456	82	29.1	–	0	0	Knife	6.0	63.0	63	6	6	21%
Floor 32		1538	82	30.8	–	0	0	Knife	6.0	63.0	63	6	6	20%
Floor 31	1826	1620	82	32.4	–	0	0	Knife	6.0	63.0	63	6	6	19%
Floor 30		1702	82	34.0	–	0	0	Knife	6.0	63.0	63	6	6	18%
Floor 29	2010	1785	83	35.7	–	0	0	Knife	6.0	63.0	63	6	6	17%
Floor 28		1868	82	37.4	–	0	0	Knife	6.0	63.0	63	6	6	16%
Floor 27	2194	1950	83	39.0	–	0	0	Knife	6.0	63.0	63	6	6	15%
Floor 26		2033	83	40.7	–	0	0	Knife	6.0	63.0	63	6	6	15%
Floor 25	2378	2116	83	42.3	–	0	0	Knife	6.0	63.0	63	6	6	14%
Floor 24		2199	83	44.0	–	0	0	Knife	6.0	63.0	63	6	6	14%
Floor 23	2562	2303	104	46.1	–	0	0	Knife	6.0	63.0	63	6	6	13%
Floor 22		2407	104	48.1	–	0	0	Knife	6.0	63.0	63	6	6	12%
Floor 21	2792	2512	105	50.2	–	0	0	Knife	6.0	63.0	63	6	6	12%
Floor 20		2597	85	51.9	–	0	0	Knife	6.0	63.0	63	6	6	12%
Floor 19	2999	2682	85	53.6	–	0	0	Knife	6.0	63.0	63	6	6	11%
Floor 18		2766	85	55.3	–	0	0	Knife	6.0	63.0	63	6	6	11%
Floor 17	3183	2851	85	57.0	–	0	0	Knife	6.0	63.0	63	6	6	11%
Floor 16		2936	85	58.7	–	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 15	3367	3021	85	60.4	–	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 14		3106	85	62.1	–	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 13	3551	3192	86	63.8	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 12		3278	85	65.6	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 11	3735	3363	86	67.3	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 10		3450	87	69.0	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 9	3919	3537	87	70.7	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 8		3623	86	72.5	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 7	3754	3306	-317	66.1	Knife	6.0	63.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		3306	0	66.1	–	0	0	Fin	86.5	86.5	87	87	87	131%
Floor 5	4120	3219	-88	64.4	–	0	0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		3371	153	67.4	–	0	0	Fin	86.5	88.5	87	87	87	128%
Floor 3	4568	–	–	–	–	0	0	Fin	272.8	276.8	–	–	–	–
Floor 2		–	–	–	–	–	–	–	–	–	–	–	–	–

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

C6.14 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 64 (EAST – WEST)

Column 64 - East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		184	184	3.7	Truss Cnx	NC	NC	Header	78.9	295.1	NC	NC	NC	>100
Floor 47	152	254	70	5.1	Header	43.5	241.3	Header	29.3	189.3	233	271	233	4587%
Floor 46		389	135	7.0	Header	49.7	269.1	Header	77.1	289.5	339	346	339	4364%
Floor 45	537	472	84	9.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1470%
Floor 44		555	82	11.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1252%
Floor 43	722	638	83	12.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1089%
Floor 42		719	82	14.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	966%
Floor 41	906	801	82	16.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	867%
Floor 40		883	81	17.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	787%
Floor 39	1090	965	82	19.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	720%
Floor 38		1046	81	20.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	664%
Floor 37	1274	1128	82	22.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	616%
Floor 36		1210	82	24.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	574%
Floor 35	1458	1292	82	25.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	538%
Floor 34		1374	82	27.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	506%
Floor 33	1642	1456	82	29.1	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 32		1538	82	30.8	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 31	1826	1620	82	32.4	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 30		1702	82	34.0	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 29	2010	1785	83	35.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	389%
Floor 28		1868	82	37.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	372%
Floor 27	2194	1950	83	39.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	356%
Floor 26		2033	83	40.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	342%
Floor 25	2378	2116	83	42.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	328%
Floor 24		2199	83	44.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	316%
Floor 23	2562	2303	104	46.1	Header	16.4	122.5	Header	20.7	144.8	161	143	143	311%
Floor 22		2407	104	48.1	Header	16.4	122.5	Header	20.7	144.8	161	143	143	297%
Floor 21	2792	2512	105	50.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	277%
Floor 20		2597	85	51.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	268%
Floor 19	2999	2682	85	53.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	259%
Floor 18		2766	85	55.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	251%
Floor 17	3183	2851	85	57.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	244%
Floor 16		2936	85	58.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	237%
Floor 15	3367	3021	85	60.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	230%
Floor 14		3106	85	62.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	224%
Floor 13	3551	3192	86	63.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	218%
Floor 12		3278	85	65.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	212%
Floor 11	3735	3363	86	67.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	207%
Floor 10		3450	87	69.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	201%
Floor 9	3919	3537	87	70.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	196%
Floor 8		3623	86	72.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	192%
Floor 7	3754	3306	-317	66.1	Header	135.7	439.8	Header	84.1	227.3	NC	NC	NC	>100
Floor 6		3306	0	66.1	Header	20.7	144.8	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 5	4120	3219	-88	64.4	Header	26.5	174.5	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		3371	153	67.4	Header	29.3	189.3	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 3	4568	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.15 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 65 (NORTH – SOUTH)

Column 65 - North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] North Connection Conn. Type	T [kip]	C [kip]	South Connection Conn. Type	T [kip]	C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only) North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		213	213	4.3	Knife	10.3	105.0	Knife	12.4	126.0	136	117	117	2759%
Floor 47	204	297	84	5.9	Knife	14.0	210.0	Knife	12.6	189.0	203	223	203	3420%
Floor 46		317	20	6.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	1099%
Floor 45	411	338	21	6.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	1021%
Floor 44		359	21	7.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	962%
Floor 43	464	380	21	7.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	908%
Floor 42		401	21	8.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	861%
Floor 41	511	421	21	8.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	819%
Floor 40		442	21	8.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	781%
Floor 39	557	463	21	9.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	746%
Floor 38		483	21	9.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	714%
Floor 37	603	504	21	10.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	684%
Floor 36		525	20	10.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	658%
Floor 35	649	545	21	10.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	633%
Floor 34		566	21	11.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	610%
Floor 33	695	587	21	11.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	588%
Floor 32		607	21	12.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	568%
Floor 31	741	628	21	12.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	549%
Floor 30		649	21	13.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	532%
Floor 29	787	669	21	13.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	515%
Floor 28		690	21	13.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	500%
Floor 27	833	711	21	14.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	485%
Floor 26		731	21	14.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	472%
Floor 25	879	752	21	15.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	459%
Floor 24		773	21	15.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	446%
Floor 23	925	799	26	16.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	432%
Floor 22		824	26	16.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	419%
Floor 21	983	850	26	17.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	406%
Floor 20		871	21	17.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	396%
Floor 19	1035	893	21	17.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	387%
Floor 18		914	21	18.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	378%
Floor 17	1081	935	21	18.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	369%
Floor 16		956	21	19.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	361%
Floor 15	1127	978	21	19.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	353%
Floor 14		999	21	20.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	345%
Floor 13	1173	1021	21	20.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	338%
Floor 12		1042	21	20.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	331%
Floor 11	1219	1064	22	21.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	324%
Floor 10		1086	22	21.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	318%
Floor 9	1265	1107	22	22.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	312%
Floor 8		1129	22	22.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	306%
Floor 7	1336	1166	37	23.3	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		1197	31	23.9	Fin	86.5	86.5	Fin	121.1	121.1	208	208	208	867%
Floor 5	1418	1012	-185	20.2	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		1031	19	20.6	Fin	86.5	86.5	Fin	121.1	121.1	208	208	208	1008%
Floor 3	1539	1578	547	31.6	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 2		1606	28	32.1	Fin	121.1	121.1	Fin	200.4	207.6	329	322	322	1001%

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

C6.16 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 65 (EAST – WEST)

Column 65 - East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection			West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]				
Roof		213	213	4.3	Knife	16.8	168.0	Header	29.3	189.3	206	197	197	4638%
Floor 47	204	297	84	5.9	Header	16.4	122.5	Header	37.9	233.8	250	160	160	2702%
Floor 46		317	20	6.3	Header	16.4	122.5	Header	20.7	144.8	161	143	143	2261%
Floor 45	411	338	21	6.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	2055%
Floor 44		359	21	7.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1938%
Floor 43	464	380	21	7.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1829%
Floor 42		401	21	8.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1734%
Floor 41	511	421	21	8.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1648%
Floor 40		442	21	8.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1571%
Floor 39	557	463	21	9.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1501%
Floor 38		483	21	9.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1437%
Floor 37	603	504	21	10.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1378%
Floor 36		526	20	10.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1324%
Floor 35	649	545	21	10.9	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 34		566	21	11.3	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 33	695	587	21	11.7	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 32		607	21	12.1	Header	16.4	122.5	Header	No Info	No Info	No info	No info	No info	-
Floor 31	741	628	21	12.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1106%
Floor 30		649	21	13.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1071%
Floor 29	787	669	21	13.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1038%
Floor 28		690	21	13.8	Header	16.4	122.5	Header	16.4	122.5	138	138	138	1007%
Floor 27	833	711	21	14.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	977%
Floor 26		731	21	14.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	950%
Floor 25	879	752	21	15.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	923%
Floor 24		773	21	15.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	899%
Floor 23	925	799	26	16.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	870%
Floor 22		824	26	16.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	843%
Floor 21	983	850	26	17.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	817%
Floor 20		871	21	17.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	797%
Floor 19	1035	893	21	17.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	778%
Floor 18		914	21	18.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	760%
Floor 17	1081	935	21	18.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	743%
Floor 16		956	21	19.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	726%
Floor 15	1127	978	21	19.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	711%
Floor 14		999	21	20.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	695%
Floor 13	1173	1021	21	20.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	681%
Floor 12		1042	21	20.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	667%
Floor 11	1219	1064	22	21.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	653%
Floor 10		1086	22	21.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	640%
Floor 9	1265	1107	22	22.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	627%
Floor 8		1129	22	22.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	615%
Floor 7	1336	1166	37	23.3	Header	16.4	122.5	Header	12.2	100.2	117	135	117	500%
Floor 6		1197	31	23.9	Header	16.4	122.5	Header	17.9	129.9	146	140	140	586%
Floor 5	1418	1012	-185	20.2	Header	16.4	122.5	Header	42.2	256.1	273	165	165	813%
Floor 4		1031	19	20.6	Header	16.4	122.5	Header	12.2	100.2	117	135	117	568%
Floor 3	1539	1578	547	31.6	Header	26.5	174.5	Header	33.6	211.0	238	208	208	659%
Floor 2		1606	28	32.1	Header	16.4	122.5	Header	25.0	167.0	182	148	148	459%

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

C6.17 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 66 (NORTH – SOUTH)

Column 66 - North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] North Connection Conn. Type	North Connection T [kip]	North Connection C [kip]	South Connection Conn. Type	South Connection T [kip]	South Connection C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only) North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		32	32	0.6	Knife	8.1	84.0		0	0	8	84	8	1259%
Floor 47	149	77	44	1.5	M-Conn	NC	NC	–	0	0	NC	NC	NC	>100
Floor 46		119	42	2.4	Knife	6.0	63.0	–	0	0	6	63	6	253%
Floor 45	339	170	51	3.4	Knife	6.0	63.0	–	0	0	6	63	6	177%
Floor 44		220	51	4.4	Knife	6.0	63.0	–	0	0	6	63	6	136%
Floor 43	455	270	50	5.4	Knife	6.0	63.0	–	0	0	6	63	6	111%
Floor 42		320	51	6.4	Knife	6.0	63.0	–	0	0	6	63	6	94%
Floor 41	567	370	50	7.4	Knife	6.0	63.0	–	0	0	6	63	6	81%
Floor 40		421	51	8.4	Knife	6.0	63.0	–	0	0	6	63	6	71%
Floor 39	679	472	51	9.4	Knife	6.0	63.0	–	0	0	6	63	6	64%
Floor 38		523	51	10.5	Knife	6.0	63.0	–	0	0	6	63	6	57%
Floor 37	791	575	51	11.5	Knife	6.0	63.0	–	0	0	6	63	6	52%
Floor 36		625	51	12.5	Knife	6.0	63.0	–	0	0	6	63	6	48%
Floor 35	903	676	51	13.5	Knife	6.0	63.0	–	0	0	6	63	6	44%
Floor 34		727	51	14.5	Knife	6.0	63.0	–	0	0	6	63	6	41%
Floor 33	1015	779	51	15.6	Knife	6.0	63.0	–	0	0	6	63	6	39%
Floor 32		830	52	16.6	Knife	6.0	63.0	–	0	0	6	63	6	36%
Floor 31	1127	882	52	17.6	Knife	6.0	63.0	–	0	0	6	63	6	34%
Floor 30		933	51	18.7	Knife	6.0	63.0	–	0	0	6	63	6	32%
Floor 29	1239	985	52	19.7	Knife	6.0	63.0	–	0	0	6	63	6	31%
Floor 28		1037	52	20.7	Knife	6.0	63.0	–	0	0	6	63	6	29%
Floor 27	1351	1089	52	21.8	Knife	6.0	63.0	–	0	0	6	63	6	28%
Floor 26		1140	51	22.8	Knife	6.0	63.0	–	0	0	6	63	6	26%
Floor 25	1463	1191	51	23.8	Knife	6.0	63.0	–	0	0	6	63	6	25%
Floor 24		1243	52	24.9	Knife	6.0	63.0	–	0	0	6	63	6	24%
Floor 23	1575	1307	64	26.1	Knife	6.0	63.0	–	0	0	6	63	6	23%
Floor 22		1371	64	27.4	Knife	6.0	63.0	–	0	0	6	63	6	22%
Floor 21	1715	1436	64	28.7	Knife	6.0	63.0	–	0	0	6	63	6	21%
Floor 20		1489	53	29.8	Knife	6.0	63.0	–	0	0	6	63	6	20%
Floor 19	1841	1541	53	30.8	Knife	6.0	63.0	–	0	0	6	63	6	19%
Floor 18		1594	52	31.9	Knife	6.0	63.0	–	0	0	6	63	6	19%
Floor 17	1953	1646	52	32.9	Knife	6.0	63.0	–	0	0	6	63	6	18%
Floor 16		1699	53	34.0	Knife	6.0	63.0	–	0	0	6	63	6	18%
Floor 15	2065	1752	53	35.0	Knife	6.0	63.0	–	0	0	6	63	6	17%
Floor 14		1805	53	36.1	Knife	6.0	63.0	–	0	0	6	63	6	17%
Floor 13	2177	1858	53	37.2	Knife	6.0	63.0	–	0	0	6	63	6	16%
Floor 12		1911	53	38.2	Knife	6.0	63.0	–	0	0	6	63	6	16%
Floor 11	2289	1964	53	39.3	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 10		2023	59	40.5	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 9	2401	2077	53	41.5	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 8		2130	53	42.6	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 7	2533	2020	-110	40.4	Truss Cnx	NC	NC	Knife	6.0	63.0	NC	NC	NC	>100
Floor 6		2076	56	41.5	Fin	121.1	121.1	Fin	103.8	103.8	225	225	225	542%
Floor 5	2769	1718	-358	34.4	Truss Cnx	NC	NC	–	0	0	NC	NC	NC	>100
Floor 4		1706	-13	34.1	Fin	121.1	121.1	–	0	0	121	121	121	355%
Floor 3	3031	2120	415	42.4	Truss Cnx	NC	NC	–	0	0	NC	NC	NC	>100
Floor 2		2390	270	47.8	Fin	200.4	207.6	–	0	0	200	208	200	419%

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

C6.18 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 66 (EAST – WEST)

Column 66 - East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		32	32	0.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	21512%
Floor 47	149	77	44	1.5	Header	25.0	167.0	Header	25.0	167.0	192	192	192	12548%
Floor 46		119	42	2.4	Header	29.3	189.3	Header	29.3	189.3	219	219	219	9202%
Floor 45	339	170	51	3.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	3441%
Floor 44		220	51	4.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	2649%
Floor 43	455	270	50	5.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	2163%
Floor 42		320	51	6.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	1820%
Floor 41	567	370	50	7.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	1576%
Floor 40		421	51	8.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	1385%
Floor 39	679	472	51	9.4	Header	12.2	100.2	Header	16.4	122.5	135	117	117	1236%
Floor 38		523	51	10.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	1115%
Floor 37	791	575	51	11.5	Header	12.2	100.2	Header	16.4	122.5	195	117	117	1015%
Floor 36		625	51	12.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	933%
Floor 35	903	676	51	13.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	863%
Floor 34		727	51	14.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	802%
Floor 33	1015	779	51	15.6	Header	12.2	100.2	Header	16.4	122.5	135	117	117	749%
Floor 32		830	52	16.6	Header	12.2	100.2	Header	16.4	122.5	135	117	117	703%
Floor 31	1127	882	52	17.6	Header	12.2	100.2	Header	16.4	122.5	135	117	117	662%
Floor 30		933	51	18.7	Header	12.2	100.2	Header	16.4	122.5	135	117	117	625%
Floor 29	1239	985	52	19.7	Header	12.2	100.2	Header	16.4	122.5	135	117	117	597%
Floor 28		1037	52	20.7	Header	12.2	100.2	Header	16.4	122.5	135	117	117	563%
Floor 27	1351	1089	52	21.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	536%
Floor 26		1140	51	22.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	512%
Floor 25	1463	1191	51	23.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	490%
Floor 24		1243	52	24.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	469%
Floor 23	1575	1307	64	26.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	531%
Floor 22		1371	64	27.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	507%
Floor 21	1715	1436	64	28.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	484%
Floor 20		1489	53	29.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	392%
Floor 19	1841	1541	53	30.8	Header	12.2	100.2	Header	16.4	122.5	135	117	117	378%
Floor 18		1594	52	31.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	368%
Floor 17	1953	1646	52	32.9	Header	12.2	100.2	Header	16.4	122.5	135	117	117	354%
Floor 16		1699	53	34.0	Header	12.2	100.2	Header	16.4	122.5	135	117	117	343%
Floor 15	2065	1752	53	35.0	Header	12.2	100.2	Header	16.4	122.5	135	117	117	333%
Floor 14		1805	53	36.1	Header	12.2	100.2	Header	12.2	100.2	112	112	112	311%
Floor 13	2177	1858	53	37.2	Header	12.2	100.2	Header	16.4	122.5	135	117	117	314%
Floor 12		1911	53	38.2	Header	12.2	100.2	Header	16.4	122.5	135	117	117	305%
Floor 11	2289	1964	53	39.3	Header	12.2	100.2	Header	16.4	122.5	135	117	117	297%
Floor 10		2023	59	40.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	288%
Floor 9	2401	2077	53	41.5	Header	12.2	100.2	Header	16.4	122.5	135	117	117	281%
Floor 8		2130	53	42.6	Header	12.2	100.2	Header	16.4	122.5	135	117	117	274%
Floor 7	2533	2020	-110	40.4	Header	16.4	122.5	Header	72.1	194.9	NC	195	NC	>100
Floor 6		2076	56	41.5	Header	12.2	100.2	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 5	2769	1718	-358	34.4	Knife	14.1	207.6	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		1706	-13	34.1	Knife	8.1	84.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 3	3031	2120	415	42.4	Knife	10.2	105.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 2		2390	270	47.8	Knife	8.1	84.0	Truss Cnx	NC	NC	NC	NC	NC	>100

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

C6.19 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 67 (NORTH – SOUTH)

Column 67 - North-South															
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		118	118	2.4		0	0	Knife	6.0	63.0	63	6	6	256%
Floor 47	126	184	66	3.7	–	0	0	Knife	14.0	210.0	210	14	14	382%
Floor 46		356	172	7.1	–	0	0	Knife	6.0	63.0	63	6	6	85%
Floor 45	566	466	111	9.3	–	0	0	Knife	10.3	105.0	105	10	10	110%
Floor 44		569	103	11.4	–	0	0	Knife	6.0	63.0	63	6	6	53%
Floor 43	784	653	85	13.1	–	0	0	Knife	6.0	63.0	63	6	6	46%
Floor 42		756	102	15.1	–	0	0	Knife	6.0	63.0	63	6	6	40%
Floor 41	1002	845	90	16.9	–	0	0	Knife	6.0	63.0	63	6	6	36%
Floor 40		947	102	18.9	–	0	0	Knife	6.0	63.0	63	6	6	32%
Floor 39	1220	1050	103	21.0	–	0	0	Knife	6.0	63.0	63	6	6	29%
Floor 38		1153	103	23.1	–	0	0	Knife	6.0	63.0	63	6	6	26%
Floor 37	1438	1269	116	25.4	–	0	0	Seated	42	42	42	42	42	165%
Floor 36		1360	91	27.2	–	0	0	Knife	6.0	63.0	63	6	6	22%
Floor 35	1746	1452	92	29.0	–	0	0	Knife	6.0	63.0	63	6	6	21%
Floor 34		1543	91	30.9	–	0	0	Knife	6.0	63.0	63	6	6	19%
Floor 33	1938	1635	92	32.7	–	0	0	Knife	6.0	63.0	63	6	6	18%
Floor 32		1727	92	34.5	–	0	0	Knife	6.0	63.0	63	6	6	17%
Floor 31	2150	1822	95	36.4	–	0	0	Knife	6.0	63.0	63	6	6	16%
Floor 30		1918	96	38.4	–	0	0	Knife	12.6	189.0	189	13	13	33%
Floor 29	2622	2011	93	40.2	–	0	0	Knife	6.0	63.0	63	6	6	16%
Floor 28		2103	92	42.1	–	0	0	Knife	6.0	63.0	63	6	6	14%
Floor 27	2514	2196	93	43.9	–	0	0	Knife	6.0	63.0	63	6	6	14%
Floor 26		2288	92	45.8	–	0	0	Knife	6.0	63.0	63	6	6	13%
Floor 25	2706	2382	93	47.6	–	0	0	Knife	6.0	63.0	63	6	6	13%
Floor 24		2475	93	49.5	–	0	0	Knife	6.0	63.0	63	6	6	12%
Floor 23	2898	2592	117	51.8	–	0	0	Knife	6.0	63.0	63	6	6	12%
Floor 22		2709	116	54.2	–	0	0	Knife	6.0	63.0	63	6	6	11%
Floor 21	3138	2826	117	56.5	–	0	0	Knife	6.0	63.0	63	6	6	11%
Floor 20		2920	94	58.4	–	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 19	3354	3015	95	60.3	–	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 18		3110	95	62.2	–	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 17	3546	3205	96	64.1	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 16		3300	95	66.0	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 15	3738	3396	96	67.9	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 14		3491	95	69.8	–	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 13	3930	3587	96	71.7	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 12		3682	95	73.6	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 11	4122	3778	96	75.6	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 10		3875	96	77.5	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 9	4314	3972	97	79.4	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 8		4068	96	81.4	–	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 7	4088	3664	-404	73.3	Knife	6.0	63.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		3610	-53	72.2	–	0	0	Fin	121.1	121.1	121	121	121	168%
Floor 5	4488	3458	-152	69.2	–	0	0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		3643	185	72.9	–	0	0	Fin	86.5	86.5	87	87	87	119%
Floor 3	4943	–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 2		–	–	–	–	–	–	–	–	–	–	–	–	–

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

C6.20 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 67 (EAST – WEST)

Column 67 - East-West

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		118	118	2.4	Header	42.2	256.1	Header	20.7	144.8	187	277	187	7952%
Floor 47	126	184	66	3.7	Header	29.3	188.3	Header	20.7	144.8	174	210	174	4736%
Floor 46		356	172	7.1	Header	78.0	295.1	Header	68.0	322.9	402	383	383	5380%
Floor 45	566	466	111	9.3	Header	16.4	122.5	Header	29.3	189.3	206	152	152	1620%
Floor 44		569	103	11.4	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1259%
Floor 43	784	653	85	13.1	Header	16.4	122.5	Header	25.0	167.0	183	148	148	1129%
Floor 42		756	102	15.1	Header	16.4	122.5	Header	20.7	144.8	161	143	143	948%
Floor 41	1002	845	90	16.9	Header	16.4	122.5	Header	25.0	167.0	183	148	148	873%
Floor 40		947	102	18.9	Header	16.4	122.5	Header	20.7	144.8	161	143	143	756%
Floor 39	1220	1050	103	21.0	Header	16.4	122.5	Header	25.0	167.0	183	148	148	702%
Floor 38		1153	103	23.1	Header	16.4	122.5	Header	20.7	144.8	161	143	143	621%
Floor 37	1438	1269	116	25.4	Header	16.4	122.5	Header	79.7	403.7	420	202	202	797%
Floor 36		1360	91	27.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	511%
Floor 35	1746	1452	92	29.0	Header	16.4	122.5	Header	30.0	180.9	197	153	153	525%
Floor 34		1543	91	30.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	450%
Floor 33	1938	1635	92	32.7	Header	No Info	No Info	Header	30.0	180.9	No info	No info	No info	-
Floor 32		1727	92	34.5	Header	No Info	No Info	Header	16.4	122.5	No info	No info	No info	-
Floor 31	2150	1822	95	36.4	Header	No Info	No Info	Header	36.2	208.8	No info	No info	No info	-
Floor 30		1918	96	38.4	Header	No Info	No Info	Header	20.7	144.8	No info	No info	No info	
Floor 29	2622	2011	93	40.2	Header	16.4	122.5	Header	30.0	180.9	197	153	153	379%
Floor 28		2103	92	42.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	330%
Floor 27	2514	2196	93	43.9	Header	16.4	122.5	Header	30.0	180.9	197	153	153	347%
Floor 26		2288	92	45.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	304%
Floor 25	2706	2382	93	47.6	Header	16.4	122.5	Header	30.0	180.9	197	153	153	320%
Floor 24		2475	93	49.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	281%
Floor 23	2898	2592	117	51.8	Header	20.7	144.8	Header	36.2	208.8	230	181	181	349%
Floor 22		2709	116	54.2	Header	20.7	144.8	Header	20.7	144.8	165	165	165	305%
Floor 21	3138	2826	117	56.5	Header	16.4	122.5	Header	36.2	208.8	225	159	159	281%
Floor 20		2920	94	58.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	238%
Floor 19	3354	3015	95	60.3	Header	16.4	122.5	Header	30.0	180.9	197	153	153	253%
Floor 18		3110	95	62.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	223%
Floor 17	3546	3205	96	64.1	Header	16.4	122.5	Header	30.0	180.8	197	153	153	238%
Floor 16		3300	95	66.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	211%
Floor 15	3738	3396	96	67.9	Header	16.4	122.5	Header	30.0	180.9	197	153	153	225%
Floor 14		3491	95	69.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	199%
Floor 13	3930	3587	96	71.7	Header	16.4	122.5	Header	30.0	180.9	197	153	153	213%
Floor 12		3682	95	73.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	189%
Floor 11	4122	3778	96	75.6	Header	16.4	122.5	Header	30.0	180.9	197	153	153	202%
Floor 10		3875	96	77.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	179%
Floor 9	4314	3972	97	79.4	Header	16.4	122.5	Header	30.0	180.9	197	153	153	192%
Floor 8		4068	96	81.4	Header	16.4	122.5	Header	16.4	122.5	NC	NC	NC	>100
Floor 7	4088	3664	-404	73.3	Header Pl	84.1	318.3	Header Pl	147.7	399.0	NC	NC	NC	>100
Floor 6		3610	-53	72.2	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 5	4488	3458	-152	69.2	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		3643	185	72.9	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 3	4943	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.21 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 68 (NORTH – SOUTH)

Column 68 - North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] North Connection Conn. Type	North Connection T [kip]	North Connection C [kip]	South Connection Conn. Type	South Connection T [kip]	South Connection C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only) North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		141	141	2.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	2445%
Floor 47	77	238	97	4.8	Knife	14.0	210.0	Knife	12.4	126.0	140	222	140	2945%
Floor 46		303	65	6.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	1139%
Floor 45	390	365	62	7.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	940%
Floor 44		414	49	8.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	834%
Floor 43	513	465	51	9.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	743%
Floor 42		513	49	10.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	672%
Floor 41	625	564	51	11.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	612%
Floor 40		612	49	12.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	564%
Floor 39	737	663	50	13.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	521%
Floor 38		711	49	14.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	485%
Floor 37	849	774	63	15.5	Seated	42	42	Knife	6.0	63.0	105	48	48	310%
Floor 36		816	42	16.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	423%
Floor 35	986	860	43	17.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	401%
Floor 34		902	42	18.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	383%
Floor 33	1050	946	44	18.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	365%
Floor 32		988	42	19.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	349%
Floor 31	1174	1039	51	20.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	332%
Floor 30		1094	55	21.9	Knife	17.6	189.0	Knife	17.6	189.0	202	202	202	921%
Floor 29	1404	1126	31	22.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	307%
Floor 28		1156	30	23.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	299%
Floor 27	1472	1188	32	23.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	291%
Floor 26		1218	30	24.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	283%
Floor 25	1540	1249	32	25.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	278%
Floor 24		1280	30	25.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	270%
Floor 23	1608	1319	39	26.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	262%
Floor 22		1356	37	27.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	254%
Floor 21	1692	1395	39	27.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	247%
Floor 20		1425	30	28.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	242%
Floor 19	1768	1457	32	29.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	237%
Floor 18		1488	31	29.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	232%
Floor 17	1836	1520	32	30.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	227%
Floor 16		1551	31	31.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	223%
Floor 15	1904	1583	32	31.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	218%
Floor 14		1614	31	32.3	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	214%
Floor 13	1972	1646	31	32.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	210%
Floor 12		1677	31	33.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	206%
Floor 11	2070	1710	33	34.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	202%
Floor 10		1741	31	34.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	198%
Floor 9	2108	1774	33	35.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	194%
Floor 8		1806	31	36.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	191%
Floor 7	2201	1782	-23	35.6	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		1824	41	36.5	Fin	86.5	86.5	Fin	121.1	121.1	208	208	208	569%
Floor 5	2328	1425	-399	28.5	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		1436	11	28.7	Fin	88.5	88.5	Fin	121.1	121.1	208	208	208	723%
Floor 3	2473	1958	522	39.2	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 2		1987	29	39.7	Fin	121.1	121.1	Fin	200.4	207.6	329	322	322	809%

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

C6.22 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 68 (EAST – WEST)

Column 68 – East-West

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] West Connection Conn. Type	West Connection T [kip]	West Connection C [kip]	East Connection Conn. Type	East Connection T [kip]	East Connection C [kip]	Total Bracing Capacity in the East-West Direction (Provided by the Connections Only) West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		141	141	2.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	4973%
Floor 47	77	238	97	4.8	Header	25.0	167.0	Header	20.7	144.8	170	188	170	3571%
Floor 46		303	65	6.1	Header	20.7	144.8	Header	20.7	144.8	165	165	165	2731%
Floor 45	390	365	62	7.3	Header	16.4	122.5	Header	29.3	189.3	206	152	152	2080%
Floor 44		414	49	8.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1679%
Floor 43	513	465	51	9.3	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1541%
Floor 42		513	49	10.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1354%
Floor 41	625	564	51	11.3	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1270%
Floor 40		612	49	12.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1135%
Floor 39	737	663	50	13.3	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1081%
Floor 38		711	49	14.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	976%
Floor 37	849	774	63	15.5	Header	16.4	122.5	Header	42.2	256.1	273	165	165	1064%
Floor 36		816	42	16.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	851%
Floor 35	986	860	43	17.2	Header	No Info	No Info	Header	30.0	180.9	No info	No info	No info	–
Floor 34		902	42	18.0	Header	No Info	No Info	Header	16.4	122.5	No info	No info	No info	–
Floor 33	1050	946	44	18.9	Header	No Info	No Info	Header	30.0	180.9	No info	No info	No info	–
Floor 32		988	42	19.8	Header	No Info	No Info	Header	16.4	122.5	No info	No info	No info	–
Floor 31	1174	1039	51	20.8	Header	16.4	122.5	Header	29.3	189.3	206	152	152	731%
Floor 30		1094	55	21.9	Header	16.4	122.5	Header	42.2	256.1	273	165	165	752%
Floor 29	1404	1126	31	22.5	Header	16.4	122.5	Header	30.0	180.9	197	153	153	677%
Floor 28		1156	30	23.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	601%
Floor 27	1472	1188	32	23.8	Header	16.4	122.5	Header	30.0	180.9	197	153	153	642%
Floor 26		1218	30	24.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	570%
Floor 25	1540	1249	32	25.0	Header	16.4	122.5	Header	30.0	180.9	197	153	153	610%
Floor 24		1280	30	25.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	543%
Floor 23	1608	1319	39	26.4	Header	16.4	122.5	Header	30.0	180.9	197	153	153	578%
Floor 22		1356	37	27.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	512%
Floor 21	1692	1395	39	27.9	Header	16.4	122.5	Header	30.0	180.9	197	153	153	547%
Floor 20		1425	30	28.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	487%
Floor 19	1768	1457	32	29.1	Header	16.4	122.5	Header	30.0	180.9	197	153	153	523%
Floor 18		1488	31	29.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	467%
Floor 17	1836	1520	32	30.4	Header	16.4	122.5	Header	30.0	180.8	197	153	153	502%
Floor 16		1551	31	31.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	448%
Floor 15	1904	1583	32	31.7	Header	16.4	122.5	Header	30.0	180.9	197	153	153	482%
Floor 14		1614	31	32.3	Header	12.2	100.2	Header	12.2	100.2	112	112	112	348%
Floor 13	1972	1646	31	32.9	Header	16.4	122.5	Header	30.0	162.4	179	153	153	463%
Floor 12		1677	31	33.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	414%
Floor 11	2070	1710	33	34.2	Header	16.4	122.5	Header	30.0	143.8	160	153	153	446%
Floor 10		1741	31	34.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	399%
Floor 9	2108	1774	33	35.5	Header	16.4	122.5	Header	30.0	162.4	179	153	153	430%
Floor 8		1806	31	36.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	385%
Floor 7	2201	1782	-23	35.6	Header	12.2	100.2	Header	20.7	144.8	157	121	121	339%
Floor 6		1824	41	36.5	Header	17.9	129.9	Header	17.9	129.9	148	148	148	405%
Floor 5	2326	1425	-399	28.5	Header	42.2	256.1	Header	42.2	256.1	298	298	298	1047%
Floor 4		1436	11	28.7	Header	12.2	100.2	Header	12.2	100.2	112	112	112	391%
Floor 3	2473	1958	522	39.2	Header	33.6	211.6	Header	33.6	211.6	245	245	245	626%
Floor 2		1987	29	39.7	Header	25.0	167.0	Header	25.0	167.0	192	192	192	483%

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

C6.23 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 69 (NORTH – SOUTH)

Column 69 - North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		42	42	0.8	Knife	6.0	63.0		0	0	6	63	6	713%
Floor 47	57	84	42	1.7	Knife	12.4	126.0	–	0	0	12	126	12	739%
Floor 46		136	52	2.7	Knife	6.0	63.0	–	0	0	6	63	6	222%
Floor 45	346	208	72	4.2	Knife	6.0	63.0	–	0	0	6	63	6	145%
Floor 44		275	67	5.5	Knife	6.0	63.0	–	0	0	6	63	6	109%
Floor 43	501	331	56	6.6	Knife	6.0	63.0	–	0	0	6	63	6	91%
Floor 42		398	67	8.0	Knife	6.0	63.0	–	0	0	6	63	6	76%
Floor 41	655	458	60	9.2	Knife	6.0	63.0	–	0	0	6	63	6	66%
Floor 40		525	67	10.5	Knife	6.0	63.0	–	0	0	6	63	6	57%
Floor 39	809	593	68	11.9	Knife	6.0	63.0	–	0	0	6	63	6	51%
Floor 38		660	68	13.2	Knife	6.0	63.0	–	0	0	6	63	6	46%
Floor 37	963	728	67	14.8	Knife	6.0	63.0	–	0	0	6	63	6	41%
Floor 36		795	67	15.9	Knife	6.0	63.0	–	0	0	6	63	6	38%
Floor 35	1117	863	67	17.3	Knife	6.0	63.0	–	0	0	6	63	6	35%
Floor 34		931	68	18.6	Knife	6.0	63.0	–	0	0	6	63	6	32%
Floor 33	1271	1000	69	20.0	Knife	6.0	63.0	–	0	0	6	63	6	30%
Floor 32		1068	69	21.4	Knife	6.0	63.0	–	0	0	6	63	6	28%
Floor 31	1425	1141	73	22.8	Knife	6.0	63.0	–	0	0	6	63	6	26%
Floor 30		1216	75	24.3	Knife	12.6	189.0	–	0	0	13	189	13	52%
Floor 29	1847	1278	62	25.8	Knife	6.0	63.0	–	0	0	6	63	6	24%
Floor 28		1340	62	26.8	Knife	6.0	63.0	–	0	0	6	63	6	22%
Floor 27	1785	1402	62	28.0	Knife	6.0	63.0	–	0	0	6	63	6	21%
Floor 26		1463	62	29.3	Knife	6.0	63.0	–	0	0	6	63	6	21%
Floor 25	1923	1525	62	30.5	Knife	6.0	63.0	–	0	0	6	63	6	20%
Floor 24		1587	62	31.7	Knife	6.0	63.0	–	0	0	6	63	6	19%
Floor 23	2061	1664	78	33.3	Knife	6.0	63.0	–	0	0	6	63	6	18%
Floor 22		1742	78	34.8	Knife	6.0	63.0	–	0	0	6	63	6	17%
Floor 21	2231	1820	78	36.4	Knife	6.0	63.0	–	0	0	6	63	6	17%
Floor 20		1882	62	37.6	Knife	6.0	63.0	–	0	0	6	63	6	16%
Floor 19	2385	1944	62	38.9	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 18		2006	62	40.1	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 17	2523	2069	62	41.4	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 16		2131	62	42.6	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 15	2661	2194	63	43.9	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 14		2257	64	45.1	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 13	2799	2321	63	46.4	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 12		2384	63	47.7	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 11	2937	2447	63	48.9	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 10		2511	64	50.2	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 9	3075	2574	64	51.5	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 8		2638	64	52.8	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 7	3223	2452	-186	49.0	Truss Cnx	NC	NC	Knife	6.0	63.0	NC	NC	NC	>100
Floor 6		2574	122	51.5	Fin	121.1	121.1	–	0	0	121	121	121	235%
Floor 5	3542	2127	-446	42.5	Truss Cnx	NC	NC	–	0	0	NC	NC	NC	>100
Floor 4		2028	-99	40.6	Fin	121.1	121.1	–	0	0	121	121	121	299%
Floor 3	3831	2677	649	53.5	Truss Cnx	NC	NC	–	0	0	NC	NC	NC	>100
Floor 2		3250	573	65.0	Fin	121.1	121.1	–	0	0	121	121	121	186%

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

C6.24 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 69 (EAST – WEST)

Column 69 - East-West

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]: West Connection: Conn. Type	West Connection: T [kip]	West Connection: C [kip]	East Connection: Conn. Type	East Connection: T [kip]	East Connection: C [kip]	Total Bracing Capacity in the East-West Direction (Provided by the Connections Only): West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		42	42	0.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	16470%
Floor 47	57	84	42	1.7	Header	25.0	167.0	Header	20.7	144.8	170	188	170	10130%
Floor 46		136	52	2.7	Header	29.3	189.3	Header	33.6	211.6	241	223	223	8217%
Floor 45	346	208	72	4.2	Header	16.4	122.5	Header	20.7	144.8	161	143	143	3447%
Floor 44		275	67	5.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	2526%
Floor 43	501	331	56	6.6	Header	16.4	122.5	Header	No info	No info	No info	No info	No info	–
Floor 42		398	67	8.0	Header	16.4	122.5	Header	No info	No info	No info	No info	No info	–
Floor 41	655	458	60	9.2	Header	16.4	122.5	Header	No info	No info	No info	No info	No info	–
Floor 40		525	67	10.5	Header	16.4	122.5	Header	No info	No info	No info	No info	No info	–
Floor 39	809	593	68	11.9	Header	16.4	122.5	Header	16.4	122.5	No info	No info	No info	–
Floor 38		660	68	13.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	1052%
Floor 37	963	728	67	14.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	954%
Floor 36		795	67	15.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	873%
Floor 35	1117	863	67	17.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	805%
Floor 34		931	68	18.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	746%
Floor 33	1271	1000	69	20.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	695%
Floor 32		1068	69	21.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	650%
Floor 31	1425	1141	73	22.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	609%
Floor 30		1216	75	24.3	Header	16.4	122.5	Header	33.6	211.6	228	156	156	642%
Floor 29	1847	1278	62	25.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	544%
Floor 28		1340	62	26.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	519%
Floor 27	1785	1402	62	28.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	496%
Floor 26		1463	62	29.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	475%
Floor 25	1923	1525	67	30.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	456%
Floor 24		1587	62	31.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	438%
Floor 23	2061	1664	78	33.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	417%
Floor 22		1742	78	34.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	399%
Floor 21	2231	1820	78	36.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	382%
Floor 20		1882	62	37.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	369%
Floor 19	2385	1944	62	38.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	357%
Floor 18		2006	62	40.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	346%
Floor 17	2523	2069	62	41.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	336%
Floor 16		2131	62	42.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	326%
Floor 15	2661	2194	63	43.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	317%
Floor 14		2257	64	45.1	Header	12.2	100.2	Header	12.2	100.2	112	112	112	249%
Floor 13	2799	2321	63	46.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	299%
Floor 12		2384	63	47.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	291%
Floor 11	2937	2447	63	48.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	284%
Floor 10		2511	64	50.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	277%
Floor 9	3075	2574	64	51.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	270%
Floor 8		2638	64	52.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	263%
Floor 7	3223	2452	-186	49.0	Header	72.1	194.9	Header	72.1	194.9	NC	NC	NC	>100
Floor 6		2574	172	51.5	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 5	3542	2127	-446	42.5	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		2028	-99	40.6	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 3	3831	2677	649	53.5	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 2		3250	573	65.0	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100

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

C6.25 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 70 (NORTH – SOUTH)

Column 70 - North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]: North Connection Conn. Type	North Connection T [kip]	North Connection C [kip]	South Connection Conn. Type	South Connection T [kip]	South Connection C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only): North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		80	80	1.6	-	0	0	Knife	6.0	63.0	63	6	6	376%
Floor 47	126	124	44	2.5	-	0	0	Knife	10.3	105.0	105	10	10	414%
Floor 46		357	233	7.1	-	0	0	Knife	12.4	126.0	126	12	12	173%
Floor 45	530	499	142	10.0	-	0	0	Knife	12.4	126.0	126	12	12	124%
Floor 44		627	128	12.5	-	0	0	Knife	12.4	126.0	126	12	12	99%
Floor 43	799	670	43	13.4	-	0	0	Knife	12.4	126.0	126	12	12	92%
Floor 42		813	144	16.3	-	0	0	Knife	12.4	126.0	126	12	12	76%
Floor 41	1067	863	49	17.3	-	0	0	Knife	12.4	126.0	126	12	12	72%
Floor 40		1014	151	20.3	-	0	0	Knife	12.4	126.0	126	12	12	61%
Floor 39	1335	1134	120	22.7	-	0	0	Knife	12.4	126.0	126	12	12	55%
Floor 38		1255	121	25.1	-	0	0	Knife	12.4	126.0	126	12	12	49%
Floor 37	1603	1388	133	27.8	-	0	0	Knife	9.8	147.0	147	10	10	35%
Floor 36		1494	106	29.9	-	0	0	Knife	10.3	105.0	105	10	10	34%
Floor 35	1977	1601	107	32.0	-	0	0	Knife	10.3	105.0	105	10	10	32%
Floor 34		1707	106	34.1	-	0	0	Knife	10.3	105.0	105	10	10	30%
Floor 33	2219	1815	107	36.3	-	0	0	Knife	10.3	105.0	105	10	10	28%
Floor 32		1921	107	38.4	-	0	0	Knife	10.3	105.0	105	10	10	27%
Floor 31	2461	2031	110	40.6	-	0	0	Knife	10.3	105.0	105	10	10	25%
Floor 30		2141	110	42.8	-	0	0	Knife	10.3	105.0	105	10	10	24%
Floor 29	2703	2249	109	45.0	-	0	0	Knife	10.3	105.0	105	10	10	23%
Floor 28		2358	108	47.2	-	0	0	Knife	10.3	105.0	105	10	10	22%
Floor 27	2945	2467	109	49.3	-	0	0	Knife	10.3	105.0	105	10	10	21%
Floor 26		2577	110	51.5	-	0	0	Knife	10.3	105.0	105	10	10	20%
Floor 25	3187	2687	110	53.7	-	0	0	Knife	10.3	105.0	105	10	10	19%
Floor 24		2795	108	55.9	-	0	0	Knife	10.3	105.0	105	10	10	18%
Floor 23	3429	2941	145	58.8	-	0	0	Knife	8.1	84.0	84	8	8	14%
Floor 22		3085	144	61.7	-	0	0	Knife	6.0	63.0	63	6	6	10%
Floor 21	3704	3230	145	64.6	-	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 20		3344	114	66.9	-	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 19	3950	3458	115	69.2	-	0	0	Knife	6.0	63.0	63	6	6	9%
Floor 18		3573	115	71.5	-	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 17	4168	3688	115	73.8	-	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 16		3809	121	76.2	-	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 15	4505	3939	130	78.8	-	0	0	Seated	42	42	42	42	42	53%
Floor 14		4040	102	80.8	-	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 13	4697	4143	103	82.9	-	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 12		4245	102	84.9	-	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 11	4889	4349	103	87.0	-	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 10		4452	104	89.0	-	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 9	5081	4556	104	91.1	-	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 8		4659	103	93.2	-	0	0	Knife	6.0	63.0	63	6	6	6%
Floor 7	4896	4270	-390	85.4	Knife	6.0	63.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		4214	-55	84.3	-	0	0	Fin	86.5	86.5	87	87	87	103%
Floor 5	5243	4104	-110	82.1	-	0	0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		4318	214	86.4	-	0	0	Fin	86.5	86.5	87	87	87	100%
Floor 3	5693	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.26 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 70 (EAST – WEST)

Column 70 - East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		80	80	1.6	Header	20.7	144.8	Header	25.0	167.0	188	170	170	10622%
Floor 47	126	124	44	2.5	Header	20.7	144.8	Header	20.7	144.8	165	165	165	6675%
Floor 46		357	233	7.1	Header	88.0	322.9	Header	61.8	300.7	389	385	385	5391%
Floor 45	530	499	142	10.0	Header	20.7	144.8	Header	29.3	189.3	210	174	174	1746%
Floor 44		627	128	12.5	Header	20.7	144.8	Header	29.3	189.3	210	174	174	1389%
Floor 43	799	670	43	13.4	Header	25.0	167.0	Header	No Info	No Info	NC	NC	NC	>100%
Floor 42		813	144	16.3	Header	20.7	144.6	Header	29.3	189.3	210	174	174	1070%
Floor 41	1067	863	49	17.3	Header	25.0	167.0	Header	No Info	No Info	NC	NC	NC	>100%
Floor 40		1014	151	20.3	Header	20.7	144.8	Header	29.3	189.3	210	174	174	859%
Floor 39	1335	1134	120	22.7	Header	25.0	167.0	Header	29.3	189.3	214	196	196	866%
Floor 38		1255	121	25.1	Header	20.7	144.8	Header	29.3	189.3	210	174	174	694%
Floor 37	1603	1388	133	27.8	Header	79.7	403.7	Header	29.3	189.3	269	433	269	969%
Floor 36		1494	106	29.9	Header	16.4	122.5	Header	29.3	189.3	208	152	152	508%
Floor 35	1977	1601	107	32.0	Header	30.0	180.9	Header	29.3	189.3	219	210	210	657%
Floor 34		1707	106	34.1	Header	16.4	122.5	Header	29.3	189.3	206	152	152	445%
Floor 33	2219	1815	107	36.3	Header	30.0	180.9	Header	29.3	189.3	219	210	210	579%
Floor 32		1921	107	38.4	Header	16.4	122.5	Header	29.3	189.3	206	152	152	395%
Floor 31	2461	2031	110	40.6	Header	36.2	208.8	Header	29.3	189.3	226	238	226	557%
Floor 30		2141	110	42.8	Header	20.7	144.8	Header	29.3	189.3	210	174	174	407%
Floor 29	2703	2249	109	45.0	Header	30.0	180.9	Header	29.3	189.3	219	210	210	467%
Floor 28		2358	108	47.2	Header	16.4	122.5	Header	29.3	189.3	206	152	152	322%
Floor 27	2945	2467	109	49.3	Header	30.0	180.9	Header	29.3	189.3	219	210	210	426%
Floor 26		2577	110	51.5	Header	16.4	122.5	Header	29.3	189.3	206	152	152	295%
Floor 25	3187	2687	110	53.7	Header	30.0	180.9	Header	29.3	189.3	219	210	210	391%
Floor 24		2795	108	55.9	Header	16.4	122.5	Header	25.0	167.0	183	148	148	264%
Floor 23	3429	2941	145	58.8	Header	36.2	208.8	Header	25.0	167.0	203	234	203	346%
Floor 22		3085	144	61.7	Header	20.7	144.8	Header	20.7	144.8	165	165	165	268%
Floor 21	3704	3230	145	64.6	Header	36.2	208.8	Header	20.7	144.8	181	230	181	280%
Floor 20		3344	114	66.9	Header	16.4	122.5	Header	20.7	144.8	161	143	143	214%
Floor 19	3950	3458	115	69.2	Header	30.0	180.9	Header	20.7	144.8	175	202	175	253%
Floor 18		3573	115	71.5	Header	16.4	122.5	Header	20.7	144.8	161	143	143	200%
Floor 17	4168	3688	115	73.8	Header	30.0	180.9	Header	20.7	144.8	175	202	175	237%
Floor 16		3809	121	76.2	Header	16.4	122.5	Header	25.0	167.0	183	148	148	194%
Floor 15	4505	3939	130	78.8	Header	30.0	180.9	Seated	42	42	72	223	72	91%
Floor 14		4040	102	80.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	172%
Floor 13	4697	4143	103	82.9	Header	30.0	180.9	Header	16.4	122.5	153	197	153	184%
Floor 12		4245	102	84.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	164%
Floor 11	4889	4349	103	87.0	Header	30.0	180.9	Header	16.4	122.5	153	197	153	175%
Floor 10		4452	104	89.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	156%
Floor 9	5081	4556	104	91.1	Header	30.0	180.9	Header	16.4	122.5	153	197	153	167%
Floor 8		4659	103	93.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	149%
Floor 7	4896	4270	-390	85.4	Double Fin	519.0	519.0	Double Fin	276.8	276.8	796	796	796	932%
Floor 6		4214	-55	84.3	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 5	5243	4104	-110	82.1	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		4318	214	86.4	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 3	5693	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.27 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 71 (NORTH – SOUTH)

Column 71 – North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		105	105	2.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	3301%
Floor 47	77	170	65	3.4	Knife	10.3	105.0	Knife	8.1	84.0	94	113	94	2774%
Floor 46		254	84	5.1	Knife	12.4	126.0	Knife	8.1	84.0	96	134	96	1896%
Floor 45	390	334	80	6.7	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	1120%
Floor 44		398	64	8.0	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	947%
Floor 43	513	436	38	8.7	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	864%
Floor 42		516	62	10.4	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	728%
Floor 41	625	553	35	11.1	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	681%
Floor 40		646	93	12.9	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	583%
Floor 39	737	715	69	14.3	Knife	13.1	126.0	Knife	6.0	63.0	76	132	76	532%
Floor 38		783	68	15.7	Knife	13.1	126.0	Knife	6.0	63.0	76	132	76	488%
Floor 37	849	864	81	17.3	Knife	9.8	147.0	Knife	6.0	63.0	73	153	73	421%
Floor 36		919	55	18.4	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	398%
Floor 35	986	976	57	19.5	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	375%
Floor 34		1032	56	20.6	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	355%
Floor 33	1030	1089	57	21.8	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	336%
Floor 32		1146	56	22.9	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	320%
Floor 31	1174	1209	63	24.2	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	303%
Floor 30		1276	67	25.5	Knife	10.3	105.0	Knife	14.0	210.0	220	119	119	467%
Floor 29	1404	1324	49	26.5	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	277%
Floor 28		1372	48	27.4	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	267%
Floor 27	1472	1421	49	28.4	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	258%
Floor 26		1469	48	29.4	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	249%
Floor 25	1540	1518	49	30.4	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	241%
Floor 24		1566	48	31.3	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	234%
Floor 23	2545	1637	70	32.7	Knife	8.1	84.0	Knife	12.6	189.0	197	97	97	295%
Floor 22		1690	53	33.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	204%
Floor 21	2603	1744	54	34.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	198%
Floor 20		1786	42	35.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	193%
Floor 19	2708	1831	44	36.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	188%
Floor 18		1874	43	37.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	184%
Floor 17	2802	1919	45	38.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	180%
Floor 16		1968	49	39.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	175%
Floor 15	3015	2026	59	40.5	Seated	42	42	Knife	6.0	63.0	105	48	48	118%
Floor 14		2058	31	41.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	168%
Floor 13	3083	2089	31	41.8	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	165%
Floor 12		2121	31	42.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	163%
Floor 11	3151	2154	33	43.1	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	160%
Floor 10		2185	31	43.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	158%
Floor 9	3219	2218	33	44.4	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	156%
Floor 8		2250	32	45.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	153%
Floor 7	3337	2175	-74	43.5	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		2213	38	44.3	Fin	86.5	86.5	Fin	121.1	121.1	208	208	208	469%
Floor 5	3497	1682	-532	33.6	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		1707	25	34.1	Fin	86.5	86.5	Fin	121.1	121.1	208	208	208	608%
Floor 3	3664	2232	525	44.6	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 2		2262	30	45.2	Fin	121.1	121.1	Fin	200.4	207.6	329	322	322	711%

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

C6.28 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 71 (EAST – WEST)

Column 71 – East-West

Floor Level	As-built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] West Connection Conn. Type	West Connection T [kip]	West Connection C [kip]	East Connection Conn. Type	East Connection T [kip]	East Connection C [kip]	Total Bracing Capacity in the East-West Direction (Provided by the Connections Only) West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		105	105	2.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	6846%
Floor 47	77	170	65	3.4	Header	20.7	144.8	Header	33.6	211.6	232	178	178	5250%
Floor 46		254	84	5.1	Header	25.0	167.0	Header	20.7	144.8	170	180	170	3340%
Floor 45	390	334	80	6.7	Header	29.3	189.3	Header	20.7	144.8	174	210	174	2605%
Floor 44		398	64	8.0	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1799%
Floor 43	513	436	38	8.7	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1897%
Floor 42		516	62	10.4	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1362%
Floor 41	625	553	35	11.1	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1495%
Floor 40		646	93	12.9	Header	16.4	122.5	Header	20.7	144.8	161	143	143	1108%
Floor 39	737	715	69	14.3	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1157%
Floor 38		783	68	15.7	Header	16.4	122.5	Header	20.7	144.8	161	143	143	914%
Floor 37	849	864	81	17.3	Header	42.2	256.1	Header	20.7	144.8	187	277	187	1082%
Floor 36		919	55	18.4	Header	16.4	122.5	Header	20.7	144.8	161	143	143	779%
Floor 35	986	976	57	19.5	Header	30.0	180.9	Header	20.7	144.8	175	202	175	895%
Floor 34		1032	56	20.6	Header	16.4	122.5	Header	20.7	144.0	161	143	143	694%
Floor 33	1030	1089	57	21.8	Header	30.0	180.9	Header	20.7	144.8	175	202	175	802%
Floor 32		1146	56	22.9	Header	16.4	122.5	Header	20.7	144.8	161	143	143	625%
Floor 31	1174	1209	63	24.2	Header	29.3	189.3	Header	20.7	144.8	174	210	174	720%
Floor 30		1276	67	25.5	Header	42.2	256.1	Header	20.7	144.8	187	277	187	733%
Floor 29	1404	1324	49	26.5	Header	30.0	180.9	Header	20.7	144.8	175	202	175	660%
Floor 28		1372	48	27.4	Header	16.4	122.5	Header	20.7	144.8	161	143	143	522%
Floor 27	1472	1421	49	28.4	Header	30.0	180.9	Header	20.7	144.8	175	202	175	615%
Floor 26		1469	48	29.4	Header	16.4	122.5	Header	20.7	144.8	161	143	143	488%
Floor 25	1540	1518	49	30.4	Header	30.0	180.9	Header	20.7	144.8	175	202	175	578%
Floor 24		1566	48	31.3	Header	16.4	122.5	Header	20.7	144.8	161	143	143	457%
Floor 23	2545	1637	70	32.7	Header	30.0	180.9	Header	42.2	256.1	286	223	223	682%
Floor 22		1690	53	33.8	Header	16.4	122.5	Header	20.7	144.8	161	143	143	424%
Floor 21	2603	1744	54	34.9	Header	30.0	180.9	Header	20.7	144.0	175	202	175	501%
Floor 20		1786	42	35.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	389%
Floor 19	2708	1831	44	36.6	Header	30.0	180.9	Header	16.4	122.5	153	197	153	417%
Floor 18		1874	43	37.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	371%
Floor 17	2802	1919	45	38.4	Header	30.0	180.9	Header	16.4	122.5	153	197	153	397%
Floor 16		1968	49	39.4	Header	16.4	122.5	Header	25.0	167.0	183	148	148	375%
Floor 15	3015	2026	59	40.5	Header	30.0	180.9	Header	33.6	211.6	242	215	215	529%
Floor 14		2058	31	41.2	Header	12.2	100.2	Header	16.4	122.5	135	117	117	283%
Floor 13	3083	2089	31	41.8	Header	30.0	180.9	Header	16.4	122.5	153	197	153	365%
Floor 12		2121	31	42.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	328%
Floor 11	3151	2154	33	43.1	Header	30.0	180.9	Header	16.4	122.5	153	197	153	354%
Floor 10		2185	31	43.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	318%
Floor 9	3219	2218	33	44.4	Header	30.0	180.9	Header	16.4	122.5	153	197	153	344%
Floor 8		2250	32	45.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	309%
Floor 7	3337	2175	-74	43.5	Header	20.7	144.8	Header	16.4	122.5	143	161	143	329%
Floor 6		2213	38	44.3	Header	17.9	129.9	Header	16.4	122.5	140	146	140	317%
Floor 5	3497	1682	-532	33.6	Header	42.2	256.1	Header	42.2	256.1	298	298	298	887%
Floor 4		1707	25	34.1	Header	7.9	77.9	Header	33.6	211.6	219	112	112	327%
Floor 3	3664	2232	525	44.6	Header	33.6	211.6	Header	33.6	211.6	245	245	245	549%
Floor 2		2262	30	45.2	Header	25.0	167.0	Header	16.4	122.5	148	183	148	326%

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

C6.29 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 72 (NORTH – SOUTH)

Column 72 - North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		39	39	0.8	Knife	6.0	63.0	–	0	0	6	63	6	785%
Floor 47	87	85	46	1.7	Knife	8.1	84.0	–	0	0	8	84	8	477%
Floor 46		170	84	3.4	Knife	8.1	84.0	–	0	0	8	84	8	240%
Floor 45	485	253	84	5.1	Knife	6.0	63.0	–	0	0	6	63	6	119%
Floor 44		331	78	6.6	Knife	6.0	63.0	–	0	0	6	63	6	91%
Floor 43	674	361	30	7.2	Knife	6.0	63.0	–	0	0	6	63	6	83%
Floor 42		454	93	9.1	Knife	6.0	63.0	–	0	0	6	63	6	66%
Floor 41	862	493	39	9.9	Knife	6.0	63.0	–	0	0	6	63	6	61%
Floor 40		597	104	11.9	Knife	6.0	63.0	–	0	0	6	63	6	50%
Floor 39	1050	676	79	13.5	Knife	6.0	63.0	–	0	0	6	63	6	44%
Floor 38		756	80	15.1	Knife	6.0	63.0	–	0	0	6	63	6	40%
Floor 37	1238	836	80	16.7	Knife	6.0	63.0	–	0	0	6	63	6	36%
Floor 36		916	80	18.3	Knife	6.0	63.0	–	0	0	6	63	6	33%
Floor 35	1426	997	81	19.9	Knife	6.0	63.0	–	0	0	6	63	6	30%
Floor 34		1078	81	21.6	Knife	6.0	63.0	–	0	0	6	63	6	28%
Floor 33	1614	1159	81	23.2	Knife	6.0	63.0	–	0	0	6	63	6	26%
Floor 32		1241	82	24.8	Knife	6.0	63.0	–	0	0	6	63	6	24%
Floor 31	1802	1327	86	26.5	Knife	6.0	63.0	–	0	0	6	63	6	23%
Floor 30		1415	89	28.3	Knife	14.0	210.0	–	0	0	14	210	14	50%
Floor 29	2069	1489	73	29.8	Knife	6.0	63.0	–	0	0	6	63	6	20%
Floor 28		1562	74	31.2	Knife	6.0	63.0	–	0	0	6	63	6	19%
Floor 27	2239	1636	73	32.7	Knife	6.0	63.0	–	0	0	6	63	6	18%
Floor 26		1709	74	34.2	Knife	6.0	63.0	–	0	0	6	63	6	18%
Floor 25	2409	1783	74	35.7	Knife	6.0	63.0	–	0	0	6	63	6	17%
Floor 24		1854	71	37.1	Knife	6.0	63.0	–	0	0	6	63	6	16%
Floor 23	2579	1946	93	38.9	Knife	11.2	168.0	–	0	0	11	168	11	29%
Floor 22		2026	79	40.5	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 21	2835	2105	79	42.1	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 20		2169	64	43.4	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 19	2989	2233	64	44.7	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 18		2297	64	45.9	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 17	3127	2361	64	47.2	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 16		2425	64	48.5	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 15	3265	2489	64	49.8	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 14		2554	65	51.1	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 13	3403	2619	65	52.4	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 12		2683	65	53.7	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 11	3541	2748	65	55.0	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 10		2814	66	56.3	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 9	3679	2879	65	57.6	Knife	6.0	63.0	–	0	0	6	63	6	10%
Floor 8		2944	65	58.9	Knife	6.0	63.0	–	0	0	6	63	6	10%
Floor 7	3857	2740	-204	54.8	Truss Cnx	NC	NC	Knife	6.0	63.0	NC	NC	NC	>100
Floor 6		2856	116	57.1	Fin	121.1	121.1	–	0	0	121	121	121	212%
Floor 5	4178	2306	-550	46.1	Truss Cnx	NC	NC	–	0	0	NC	NC	NC	>100
Floor 4		2156	-150	43.1	Fin	121.1	121.1	–	0	0	121	121	121	281%
Floor 3	4460	2855	699	57.1	Truss Cnx	NC	NC	–	0	0	NC	NC	NC	>100
Floor 2		3590	735	71.8	Fin	121.1	121.1	–	0	0	121	121	121	169%

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

C6.30 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 72 (EAST – WEST)

Column 72 – East-West

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] West Connection Conn. Type	West Connection T [kip]	West Connection C [kip]	East Connection Conn. Type	East Connection T [kip]	East Connection C [kip]	Total Bracing Capacity in the East-West Direction (Provided by the Connections Only) West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		39	39	0.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	17682%
Floor 47	87	85	46	1.7	Header	20.7	144.8	Header	33.6	211.6	232	178	178	10458%
Floor 46		170	84	3.4	Header	37.3	213.4	Header	61.0	300.7	338	275	275	8100%
Floor 45	485	253	84	5.1	Header	20.7	144.8	Header	33.6	211.6	232	170	178	3510%
Floor 44		331	78	6.6	Header	16.4	122.5	Header	33.6	211.6	228	156	156	2357%
Floor 43	674	361	30	7.2	Header	No Info	No Info	Header	No Info	No Info	No info	No info	No info	–
Floor 42		454	93	9.1	Header	No Info	No Info	Header	33.6	211.6	No info	No info	No info	–
Floor 41	862	493	39	9.9	Header	No Info	No Info	Header	No Info	No Info	No info	No info	No info	–
Floor 40		597	104	11.9	Header	No Info	No Info	Header	33.6	211.6	No info	No info	No info	–
Floor 39	1050	676	79	13.5	Header	16.4	122.5	Header	33.6	211.6	228	156	156	1155%
Floor 38		756	80	15.1	Header	16.4	122.5	Header	33.6	211.6	228	156	156	1033%
Floor 37	1238	836	80	16.7	Header	16.4	122.5	Header	33.6	211.6	228	158	156	934%
Floor 36		916	80	18.3	Header	16.4	122.5	Header	33.6	211.6	228	156	156	852%
Floor 35	1426	997	81	19.9	Header	16.4	122.5	Header	33.6	211.6	228	156	156	783%
Floor 34		1078	81	21.6	Header	16.4	122.5	Header	33.6	211.6	228	156	156	724%
Floor 33	1614	1159	81	23.2	Header	16.4	122.5	Header	33.6	211.6	228	156	156	673%
Floor 32		1241	82	24.8	Header	16.4	122.5	Header	33.6	211.6	228	156	156	629%
Floor 31	1802	1327	86	26.5	Header	16.4	122.5	Header	33.6	211.6	228	156	156	588%
Floor 30		1415	89	28.3	Header	33.6	211.6	Header	33.6	211.6	245	245	245	868%
Floor 29	2069	1489	73	29.8	Header	16.4	122.5	Header	33.6	211.6	228	156	156	524%
Floor 28		1562	74	31.2	Header	16.4	122.5	Header	33.6	211.6	228	156	156	500%
Floor 27	2239	1636	73	32.7	Header	16.4	122.5	Header	33.6	211.6	228	156	156	477%
Floor 26		1709	74	34.2	Header	16.4	122.5	Header	33.6	211.6	228	156	156	457%
Floor 25	2409	1783	74	35.7	Header	16.4	122.5	Header	33.6	211.6	228	156	156	438%
Floor 24		1854	71	37.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	375%
Floor 23	2579	1946	93	38.9	Header	16.4	122.5	Header	33.6	211.6	228	156	156	401%
Floor 22		2026	79	40.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	343%
Floor 21	2835	2105	79	42.1	Header	16.4	122.5	Header	16.4	122.5	139	139	139	330%
Floor 20		2169	64	43.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	320%
Floor 19	2989	2233	64	44.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	311%
Floor 18		2297	64	45.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	302%
Floor 17	3127	2361	64	47.2	Header	16.4	122.5	Header	16.4	122.5	139	139	139	294%
Floor 16		2425	64	48.5	Header	16.4	122.5	Header	16.4	122.5	139	139	139	286%
Floor 15	3265	2489	64	49.8	Header	16.4	122.5	Header	16.4	122.5	139	139	139	279%
Floor 14		2554	65	51.1	Header	12.2	100.2	Header	16.4	122.5	135	117	117	229%
Floor 13	3403	2619	65	52.4	Header	16.4	122.5	Header	16.4	122.5	139	139	139	265%
Floor 12		2683	65	53.7	Header	16.4	122.5	Header	16.4	122.5	139	139	139	259%
Floor 11	3541	2748	65	55.0	Header	16.4	122.5	Header	16.4	122.5	139	139	139	253%
Floor 10		2814	66	56.3	Header	16.4	122.5	Header	16.4	122.5	139	139	139	247%
Floor 9	3679	2879	65	57.6	Header	16.4	122.5	Header	16.4	122.5	139	139	139	241%
Floor 8		2944	65	58.9	Header	16.4	122.5	Header	16.4	122.5	139	139	139	236%
Floor 7	3857	2740	-204	54.8	Header	72.1	194.9	Header	72.1	194.9	267	267	267	487%
Floor 6		2856	116	57.1	Truss Cnx	0	0	Truss Cnx	0	0	NC	NC	NC	>100
Floor 5	4176	2306	-550	46.1	Truss Cnx	0	0	Truss Cnx	0	0	NC	NC	NC	>100
Floor 4		2156	-150	43.1	Truss Cnx	0	0	Truss Cnx	0	0	NC	NC	NC	>100
Floor 3	4460	2855	699	57.1	Truss Cnx	0	0	Truss Cnx	0	0	NC	NC	NC	>100
Floor 2		3590	735	71.8	Truss Cnx	0	0	Truss Cnx	0	0	NC	NC	NC	>100

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

C6.31 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 73 (NORTH – SOUTH)

Column 73 – North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] North Connection Conn. Type	North Connection T [kip]	North Connection C [kip]	South Connection Conn. Type	South Connection T [kip]	South Connection C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only) North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		101	101	2.0	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	3727%
Floor 47	125	147	46	2.9	–	0	0	Knife	10.3	105.0	105	10	10	349%
Floor 46		325	178	6.5	–	0	0	Knife	12.4	126.0	126	12	12	190%
Floor 45	517	441	116	8.0	–	0	0	Knife	10.3	105.0	105	10	10	116%
Floor 44		554	113	11.1	–	0	0	Knife	10.3	105.0	105	10	10	93%
Floor 43	[illegible]	[illegible]	–	–	–	0	0	–	0	0	–	–	–	–
Floor 42		681	127	13.6	–	0	0	Knife	10.3	105.0	105	10	10	79%
Floor 41	[illegible]	[illegible]	–	–	–	0	0	–	0	0	–	–	–	
Floor 40		816	135	16.3	–	0	0	Knife	10.3	105.0	105	10	10	62%
Floor 39	1448	933	117	18.7	–	0	0	Knife	10.3	105.0	105	10	10	55%
Floor 38		1050	117	21.0	–	0	0	Knife	10.3	105.0	105	10	10	49%
Floor 37	1753	1168	118	23.4	–	0	0	Knife	10.3	105.0	105	10	10	44%
Floor 36		1287	119	25.7	–	0	0	Knife	10.3	105.0	105	10	10	40%
Floor 35	2068	1406	119	28.1	–	0	0	Knife	10.3	105.0	105	10	10	36%
Floor 34		1526	120	30.5	–	0	0	Knife	10.3	105.0	105	10	10	34%
Floor 33	2378	1646	120	32.9	–	0	0	Knife	10.3	105.0	105	10	10	31%
Floor 32		1766	120	35.3	–	0	0	Knife	10.3	105.0	105	10	10	29%
Floor 31	2680	1886	120	37.7	–	0	0	Knife	10.3	105.0	105	10	10	27%
Floor 30		2007	121	40.1	–	0	0	Knife	10.3	105.0	105	10	10	26%
Floor 29	2998	2128	121	42.6	–	0	0	Knife	10.3	105.0	105	10	10	24%
Floor 28		2249	121	45.0	–	0	0	Knife	10.3	105.0	105	10	10	23%
Floor 27	3300	2370	121	47.4	–	0	0	Knife	10.3	105.0	105	10	10	22%
Floor 26		2492	122	49.8	–	0	0	Knife	10.3	105.0	105	10	10	21%
Floor 25	3618	2614	122	52.3	–	0	0	Knife	10.3	105.0	105	10	10	20%
Floor 24		2740	126	54.8	–	0	0	Knife	12.4	126.0	126	12	12	23%
Floor 23	3928	2887	147	57.7	–	0	0	Knife	14.5	147.0	147	14	14	25%
Floor 22		3027	141	60.5	–	0	0	Knife	10.3	105.0	105	10	10	17%
Floor 21	4276	3168	141	63.4	–	0	0	Knife	10.3	105.0	105	10	10	16%
Floor 20		3284	116	65.7	–	0	0	Knife	10.3	105.0	105	10	10	16%
Floor 19	4592	3400	116	68.0	–	0	0	Knife	10.3	105.0	105	10	10	15%
Floor 18		3516	116	70.3	–	0	0	Knife	10.3	105.0	105	10	10	15%
Floor 17	4882	3633	117	72.7	–	0	0	Knife	10.3	105.0	105	10	10	14%
Floor 16		3756	124	75.1	–	0	0	Knife	12.4	126.0	126	12	12	16%
Floor 15	5291	3886	130	77.7	–	0	0	Knife	18.7	189.0	189	19	19	24%
Floor 14		3991	105	79.8	–	0	0	Knife	6.0	63.0	63	6	6	8%
Floor 13	5553	4096	105	81.9	–	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 12		4201	106	84.0	–	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 11	5815	4307	106	86.1	–	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 10		4415	108	88.3	–	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 9	6077	4521	107	90.4	–	0	0	Knife	6.0	63.0	63	6	6	7%
Floor 8		4628	107	92.6	–	0	0	Knife	6.0	63.0	63	6	6	6%
Floor 7	8411	4340	-288	86.8	Knife	6.0	63.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		4352	12	87.0	Fin	252.0	252.0	Fin	86.5	86.5	339	339	339	389%
Floor 5	9028	6615	2263	132.3	–	0.0	0.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		6796	180	135.9	–	0.0	0.0	Fin (+Angle)	86.5	86.5	87	87	87	64%
Floor 3	9710	–	–	–	–	–	–	–	–	–	–	–	–	–
Floor 2		–	–	–	–	–	–	–	–	–	–	–	–	–

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

C6.32 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 73 (EAST – WEST)

Column 73 – East-West														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		101	101	2.0	Header	20.7	144.8	Header	20.7	144.8	165	165	165	8184%
Floor 47	125	147	46	2.9	Header	20.7	144.8	Header	20.7	144.8	165	165	165	5633%
Floor 46		325	178	6.5	Header	61.0	300.7	Header	111.9	395.3	457	413	413	6340%
Floor 45	517	441	116	8.0	Header	29.3	189.3	Header	25.0	167.0	196	214	196	2220%
Floor 44		554	113	11.1	Header	29.3	189.3	Header	25.0	167.0	196	214	196	1773%
Floor 43	[illegible]	555	-	-	-	0	0	-	0	0	-	-	-	-
Floor 42		681	127	13.6	Header	29.3	189.3	Header	25.0	167.0	196	214	196	1441%
Floor 41	1110	683	-	-	-	0	0	-	0	0			-	
Floor 40		816	135	16.3	Header	29.3	189.3	Header	25.0	167.0	196	214	196	1203%
Floor 39	1448	933	117	18.7	Header	29.3	189.3	Header	25.0	167.0	196	214	196	1052%
Floor 38		1050	117	21.0	Header	29.3	189.3	Header	25.0	167.0	196	214	196	935%
Floor 37	1753	1168	118	23.4	Header	29.3	189.3	Header	25.0	167.0	196	214	196	840%
Floor 36		1287	119	25.7	Header	29.3	189.3	Header	25.0	167.0	196	214	196	763%
Floor 35	2068	1406	119	28.1	Header	29.3	189.3	Header	25.0	167.0	196	214	196	698%
Floor 34		1526	120	30.5	Header	29.3	189.3	Header	25.0	167.0	196	214	196	643%
Floor 33	2378	1646	120	32.9	Header	29.3	189.3	Header	25.0	167.0	196	214	196	597%
Floor 32		1766	120	35.3	Header	29.3	189.3	Header	25.0	167.0	196	214	196	556%
Floor 31	2680	1886	120	37.7	Header	29.3	189.3	Header	25.0	167.0	196	214	196	521%
Floor 30		2007	121	40.1	Header	29.3	189.3	Header	25.0	167.0	196	214	196	489%
Floor 29	2998	2128	121	42.6	Header	29.3	189.3	Header	25.0	167.0	196	214	196	461%
Floor 28		2249	121	45.0	Header	29.3	189.3	Header	25.0	167.0	196	214	196	437%
Floor 27	3300	2370	121	47.4	Header	29.3	189.3	Header	25.0	167.0	196	214	196	414%
Floor 26		2492	122	49.8	Header	29.3	189.3	Header	25.0	167.0	196	214	196	394%
Floor 25	3618	2614	122	52.3	Header	29.3	189.3	Header	25.0	167.0	196	214	196	376%
Floor 24		2740	126	54.8	Header	25.0	167.0	Header	33.6	211.6	237	201	201	366%
Floor 23	3928	2887	147	57.7	Header	25.0	167.0	Header	33.6	211.6	237	201	201	348%
Floor 22		3027	141	60.5	Header	20.7	144.8	Header	33.6	211.6	232	178	178	295%
Floor 21	4276	3168	141	63.4	Knife	10.3	105.0	Seated	42	42	52	147	52	82%
Floor 20		3284	116	65.7	Knife	10.3	105.0	Seated	42	42	52	147	52	80%
Floor 19	4592	3400	116	68.0	Knife	10.3	105.0	Seated	42	42	52	147	52	77%
Floor 18		3516	116	70.3	Knife	10.3	105.0	Seated	42	42	52	147	52	74%
Floor 17	4882	3633	117	72.7	Knife	10.3	105.0	Seated	42	42	52	147	52	72%
Floor 16		3756	124	75.1	Knife	12.4	126.0	Seated	42	42	54	168	54	72%
Floor 15	5291	3886	130	77.7	Seated	42	42	Seated	42	42	84	84	84	108%
Floor 14		3991	105	79.8	Knife	10.3	105.0	Seated	42	42	52	147	52	65%
Floor 13	5553	4096	105	81.9	Knife	10.3	105.0	Seated	42	42	52	147	52	64%
Floor 12		4201	106	84.0	Knife	10.3	105.0	Seated	42	42	52	147	52	62%
Floor 11	5815	4307	106	86.1	Knife	10.3	105.0	Seated	42	42	52	147	52	61%
Floor 10		4415	108	88.3	Knife	10.3	105.0	Seated	42	42	52	147	52	59%
Floor 9	6077	4521	107	90.4	Knife	10.3	105.0	Seated	42	42	52	147	52	59%
Floor 8		4628	107	92.6	Knife	10.3	105.0	Seated	42	42	52	147	52	58%
Floor 7	8411	4340	-288	86.8	Double Fin	276.8	276.8	Knife	40.9	484.4	761	318	318	366%
Floor 6		4352	12	87.0	Truss Cnx	NC	NC	No Info	No Info	No Info	No info	No info	No info	-
Floor 5	9028	6615	2263	132.3	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		6796	180	135.9	Truss Cnx	NC	NC	Knife	20.9	210.0	NC	NC	NC	>100
Floor 3	9710	-	-	-	-	-	-	-	-	-	-	-	-	-
Floor 2		-	-	-	-	-	-	-	-	-	-	-	-	-

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

C6.33 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 74 (NORTH – SOUTH)

Column 74 – North-South														
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the North-South Direction (Provided by the Connections Only)			
					North Connection			South Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		119	119	2.4	Knife	8.1	84.0	Knife	6.0	63.0	71	90	71	2993%
Floor 47	76	190	71	3.8	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	1928%
Floor 46		259	69	5.2	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	1454%
Floor 45	330	329	70	6.6	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	1112%
Floor 44		395	66	7.9	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	927%
Floor 43	561	397	-	-	-	0	0	-	0	0	-	-	-	-
Floor 42		475	80	9.5	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	771%
Floor 41	795	477	-	-	-	0	0	-	0	0				
Floor 40		562	87	11.2	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	652%
Floor 39	1027	634	72	12.7	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	578%
Floor 38		706	72	14.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	519%
Floor 37	1259	779	73	15.6	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	470%
Floor 36		853	74	17.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	430%
Floor 35	1491	927	74	18.5	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	395%
Floor 34		1001	75	20.0	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	366%
Floor 33	1723	1076	75	21.5	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	340%
Floor 32		1151	75	23.0	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	318%
Floor 31	1955	1227	75	24.5	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	299%
Floor 30		1303	76	26.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	281%
Floor 29	2187	1379	76	27.6	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	266%
Floor 28		1455	77	29.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	252%
Floor 27	2419	1535	80	30.7	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	239%
Floor 26		1615	80	32.3	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	227%
Floor 25	2651	1695	80	33.9	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	216%
Floor 24		1768	72	35.4	Knife	12.4	126.0	Knife	6.0	63.0	75	132	75	213%
Floor 23	2973	1874	106	37.5	Knife	9.8	147.0	Knife	14.0	210.0	220	161	161	430%
Floor 22		1936	62	38.7	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	189%
Floor 21	3345	1998	62	40.0	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	183%
Floor 20		2051	53	41.0	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	179%
Floor 19	3535	2103	53	42.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	174%
Floor 18		2153	49	43.1	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	170%
Floor 17	3715	2202	49	44.0	Knife	10.3	105.0	Knife	6.0	63.0	73	111	73	166%
Floor 16		2256	54	45.1	Knife	12.0	126.0	Knife	6.0	63.0	75	132	75	166%
Floor 15	3935	2317	61	46.3	Knife	12.6	189.0	Knife	6.0	63.0	76	195	76	163%
Floor 14		2359	42	47.2	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	146%
Floor 13	4105	2401	42	48.0	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	144%
Floor 12		2443	42	48.9	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	141%
Floor 11	4275	2485	42	49.7	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	139%
Floor 10		2532	47	50.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	136%
Floor 9	4445	2579	47	51.6	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	134%
Floor 8		2626	47	52.5	Knife	6.0	63.0	Knife	6.0	63.0	69	69	69	131%
Floor 7	8351	2605	-21	52.1	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 6		2649	44	53.0	Double Fin	103.8	103.8	Fin	121.1	121.1	225	225	225	425%
Floor 5	8827	4612	1963	92.2	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		4644	32	92.9	Fin (+Angle)	86.5	86.5	Fin (+Angle)	121.1	121.1	208	208	208	224%
Floor 3	9226	5171	527	103.4	Truss Cnx	NC	NC	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 2		5211	40	104.2	Fin (+Angle)	121.1	121.1	Fin (+Angle)	200.4	207.6	329	322	322	309%

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

C6.34 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 74 (EAST – WEST)

Column 74 – East-West															
Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips]						Total Bracing Capacity in the East-West Direction (Provided by the Connections Only)			
					West Connection			East Connection						
					Conn. Type	T [kip]	C [kip]	Conn. Type	T [kip]	C [kip]	West in Tension + East in Comp. [kip]	East in Tension + West in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		119	119	2.4	Header	16.4	122.5	Header	20.7	144.8	161	143	143	6025%
Floor 47	76	190	71	3.8	Header	33.6	211.6	Header	20.7	144.8	178	232	178	4694%
Floor 46		259	69	5.2	Header	20.7	144.8	Header	20.7	144.8	165	165	165	3192%
Floor 45	330	329	70	6.6	Header	20.7	144.8	Header	20.7	144.8	165	165	165	2512%
Floor 44		395	66	7.9	Header	20.7	144.8	Header	20.7	144.8	165	165	165	2095%
Floor 43	561	397	-	-	-	0	0	-	0	0	-	-	-	-
Floor 42		475	80	9.5	Header	20.7	144.6	Header	20.7	144.6	165	165	165	1741%
Floor 41	795	477	-	-	-	0	0	-	0	0				
Floor 40		562	87	11.2	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1473%
Floor 39	1027	634	72	12.7	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1305%
Floor 38		706	72	14.1	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1172%
Floor 37	1259	779	73	15.6	Header	20.7	144.8	Header	20.7	144.8	165	165	165	1062%
Floor 36		853	74	17.1	Header	20.7	144.8	Header	20.7	144.8	165	165	165	970%
Floor 35	1491	927	74	18.5	Header	20.7	144.8	Header	20.7	144.8	165	165	165	893%
Floor 34		1001	75	20.0	Header	20.7	144.8	Header	20.7	144.8	165	165	165	826%
Floor 33	1723	1076	75	21.5	Header	20.7	144.8	Header	20.7	144.8	165	165	165	769%
Floor 32		1151	75	23.0	Header	20.7	144.8	Header	20.7	144.8	165	165	165	719%
Floor 31	1955	1227	75	24.5	Header	20.7	144.8	Header	20.7	144.8	165	165	165	675%
Floor 30		1303	76	26.1	Header	20.7	144.8	Header	20.7	144.8	165	165	165	635%
Floor 29	2187	1379	76	27.6	Header	20.7	144.8	Header	20.7	144.8	165	165	165	600%
Floor 28		1456	77	29.1	Header	20.7	144.8	Header	20.7	144.8	165	165	165	569%
Floor 27	2419	1535	80	30.7	Header	20.7	144.8	Header	20.7	144.8	165	165	165	539%
Floor 26		1615	80	32.3	Header	20.7	144.8	Header	20.7	144.8	165	165	165	512%
Floor 25	2651	1695	80	33.9	Header	20.7	144.8	Header	20.7	144.8	165	165	165	488%
Floor 24		1768	72	35.4	Header	20.7	144.8	Header	33.6	211.6	232	178	178	504%
Floor 23	2973	1874	106	37.5	Header	61.1	320.1	Header	33.6	211.6	273	354	273	727%
Floor 22		1936	62	38.7	Header	20.7	144.8	Header	20.7	144.8	165	165	165	427%
Floor 21	3345	1998	62	40.0	Header	20.7	144.8	Header	20.7	144.8	165	165	165	414%
Floor 20		2051	53	41.0	Header	16.4	122.5	Header	20.7	144.8	161	143	143	349%
Floor 19	3535	2103	53	42.1	Header	16.4	122.5	Header	20.7	144.8	161	143	143	340%
Floor 18		2153	49	43.1	Header	16.4	122.5	Header	20.7	144.8	161	143	143	333%
Floor 17	3715	2202	49	44.0	Header	16.4	122.5	Header	20.7	144.8	161	143	143	325%
Floor 16		2256	54	45.1	Header	25.0	167.0	Header	20.7	144.8	170	188	170	376%
Floor 15	3935	2317	61	46.3	Header	33.6	211.6	Header	20.7	144.8	178	232	178	385%
Floor 14		2359	42	47.2	Header	16.4	122.5	Header	20.7	144.8	161	143	143	304%
Floor 13	4105	2401	42	48.0	Header	16.4	122.5	Header	20.7	144.8	161	143	143	298%
Floor 12		2443	42	48.9	Header	16.4	122.5	Header	20.7	144.8	161	143	143	293%
Floor 11	4275	2485	42	49.7	Header	16.4	122.5	Header	20.7	144.8	161	143	143	288%
Floor 10		2532	47	50.6	Header	16.4	122.5	Header	20.7	144.8	161	143	143	283%
Floor 9	4445	2579	47	51.6	Header	16.4	122.5	Header	20.7	144.8	161	143	143	278%
Floor 8		2626	47	52.5	Header	16.4	122.5	Header	20.7	144.8	161	143	143	273%
Floor 7	8351	2605	-21	52.1	Knife	8.1	84.0	Knife	14.5	147.0	155	98	98	189%
Floor 6		2649	44	53.0	Knife	8.1	84.0	–	0.0	0.0	8	84	8	15%
Floor 5	8827	4612	1963	92.2	Knife	14.0	210.0	Truss Cnx	NC	NC	NC	NC	NC	>100
Floor 4		4644	32	92.9	Knife	11.2	168.0	–	0.0	0.0	11	168	11	12%
Floor 3	9226	5171	527	103.4	Knife	11.2	168.0	–	0.0	0.0	11	168	11	11%
Floor 2		5211	40	104.2	Knife	8.1	84.0	–	0.0	0.0	8	84	8	8%

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

C6.35 INTERIOR COLUMN BRACING CAPACITY CATALOGUE – COLUMN 75 (NORTH – SOUTH)

Column 75 - North-South

Floor Level	As built Axial Column Loads (Cumulative) [Kip]	Design Axial Column Loads (Cumulative) [Kip]	Design Axial Column Load (Increments) [Kip]	Req Conn Axial Capacity (2%) [Kip]	As-Designed Axial Capacity of Bracing Connections [Kips] — North Connection: Conn. Type	North Connection: T [kip]	North Connection: C [kip]	South Connection: Conn. Type	South Connection: T [kip]	South Connection: C [kip]	Total Bracing Capacity in the North-South Direction (Provided by the Connections Only) — North in Tension + South in Comp. [kip]	South in Tension + North in Comp. [kip]	Minimum [kip]	Percentage of Req. Bracing Capacity
Roof		58	58	1.2	Knife	6.0	63.0		0	0	6	63	6	522%
Floor 47	85	105	47	2.1	Knife	6.0	63.0	–	0	0	6	63	6	280%
Floor 46		207	102	4.1	Knife	6.0	63.0	–	0	0	6	63	6	145%
Floor 45	398	279	72	5.6	Knife	6.0	63.0	–	0	0	6	63	6	108%
Floor 44		350	71	7.0	Knife	6.0	63.0	–	0	0	6	63	6	89%
Floor 43	626	351	–	–	0.0	0	0	–	0	0	–	–	–	–
Floor 42		432	82	8.6	Knife	6.0	63.0	–	0	0	6	63	6	70%
Floor 41	858	431	–	–	0.0	0	0	–	0	0	–	–	–	–
Floor 40		520	88	10.4	Knife	6.0	63.0	–	0	0	6	63	6	59%
Floor 39	1074	593	73	11.9	Knife	6.0	63.0	–	0	0	6	63	6	51%
Floor 38		666	73	13.3	Knife	6.0	63.0	–	0	0	6	63	6	45%
Floor 37	1298	739	73	14.8	Knife	6.0	63.0	–	0	0	6	63	6	41%
Floor 36		813	74	16.3	Knife	6.0	63.0	–	0	0	6	63	6	37%
Floor 35	1522	888	74	17.8	Knife	6.0	63.0	–	0	0	6	63	6	34%
Floor 34		963	75	19.3	Knife	6.0	63.0	–	0	0	6	63	6	31%
Floor 33	1746	1038	75	20.8	Knife	6.0	63.0	–	0	0	6	63	6	29%
Floor 32		1113	75	22.3	Knife	6.0	63.0	–	0	0	6	63	6	27%
Floor 31	1970	1188	75	23.8	Knife	6.0	63.0	–	0	0	6	63	6	25%
Floor 30		1264	76	25.3	Knife	6.0	63.0	–	0	0	6	63	6	24%
Floor 29	2194	1340	76	26.8	Knife	6.0	63.0	–	0	0	6	63	6	22%
Floor 28		1417	76	28.3	Knife	6.0	63.0	–	0	0	6	63	6	21%
Floor 27	2418	1496	80	29.9	Knife	6.0	63.0	–	0	0	6	63	6	20%
Floor 26		1576	80	31.5	Knife	6.0	63.0	–	0	0	6	63	6	19%
Floor 25	2642	1656	80	33.1	Knife	6.0	63.0	–	0	0	6	63	6	18%
Floor 24		1739	83	34.8	Knife	6.0	63.0	–	0	0	6	63	6	17%
Floor 23	2866	1831	92	36.6	Knife	14.0	210.0	–	0	0	14	210	14	38%
Floor 22		1922	91	38.4	Knife	6.0	63.0	–	0	0	6	63	6	16%
Floor 21	3170	2013	91	40.3	Knife	6.0	63.0	–	0	0	6	63	6	15%
Floor 20		2089	75	41.8	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 19	3372	2164	75	43.3	Knife	6.0	63.0	–	0	0	6	63	6	14%
Floor 18		2233	70	44.7	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 17	3558	2303	70	46.1	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 16		2372	69	47.4	Knife	6.0	63.0	–	0	0	6	63	6	13%
Floor 15	3744	2442	69	48.8	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 14		2512	70	50.2	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 13	3930	2582	70	51.6	Knife	6.0	63.0	–	0	0	6	63	6	12%
Floor 12		2652	70	53.0	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 11	4116	2722	70	54.4	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 10		2799	77	56.0	Knife	6.0	63.0	–	0	0	6	63	6	11%
Floor 9	4302	2876	77	57.5	Knife	6.0	63.0	–	0	0	6	63	6	10%
Floor 8		2952	77	59.0	Knife	6.0	63.0	–	0	0	6	63	6	10%
Floor 7	4528	2831	-121	56.6	Truss Cnx	0	NC	Knife	6.0	63.0	NC	NC	NC	>100
Floor 6		2952	121	59.0	Fin	121.1	121.1	–	0.0	0.0	121	121	121	205%
Floor 5	4911	2594	-358	51.9	Truss Cnx	NC	NC	–	0.0	0.0	NC	NC	NC	>100
Floor 4		2470	124	49.4	Fin (+Angle)	121.1	121.1	–	0.0	0.0	121	121	121	245%
Floor 3	5236	3363	892	67.3	Truss Cnx	NC	NC	Knife	10.3	105.0	NC	NC	NC	>100
Floor 2		3769	406	75.4	Fin (+Angle)	200.4	207.6	–	0.0	0.0	200	208	200	266%

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