STEEL BUILDING – EXTERIOR WALL INTERFACE ISSUES

JANUARY 2005

Jeff Janakus is a graduate from Colorado State University, where he received his BSCE in Civil Engineering in 1990 and a MS degree in Civil Engineering in 1992. He currently is employed by Station Casinos, Inc. in Las Vegas, Nevada. Jeff also worked for Martin/Martin, Inc., a Denver, Colorado structural engineering firm, involved with the Bank One Ballpark in Phoenix, Arizona, the Phoenix Coyotes Arena in Glendale, Arizona, U.S. Courthouse in Denver, Colorado, and the Lockheed Martin Booster Hydro Facility in Jefferson County, Colorado. He worked for Huber Hunt & Nichols, general contractor, in 1998 on the San Francisco Giants Stadium. He more recently was employed by Puma Steel, a fabricator located in Cheyenne, Wyoming.

Jeff served six years as the Chairperson for the SEAC/RMSCA Steel Liaison Committee. He is a member of AISC and DBIA.

Richard J. Huddleston, Jr. is a Project Manager and the Information Technology Manager for Zimkor LLC, a Category II AISC certified steel fabricator in the state of Colorado. Educated in Computer science and Mathematics/Physics/Drafting, Richard has experience in detailing using CAD and programming AutoCAD routines to aid in designs and detailing. He ran his first CAD program on a package called Versa CAD on an 8088 PC with no hard drive on a green screen. He has been building commercial structures and computer networks for over 20 years.

Richard is currently an Associate Member of the Structural Engineers Association of Colorado (SEAC) serving on the Structural Steel Liaison and AIA committees. While on the Steel Liaison committee, Richard helped develop position papers for Architecturally Exposed Structural Steel, Design Document Compliance and Coordination, and Structural Steel Shop Drawings.

Richard's notable projects are the stainless steel stair and escalator railing at the Disney Concert Hall in Los Angeles, the Molly Blank Asthma Research Building at the National Jewish Medical and Research Center in Denver, and the Boulder Community Hospital, which is the first LEEDS certified hospital in the nation, in Boulder, Colorado and several structural steel and aluminum projects with the Raytheon Polar Services for the National Science Foundation in the Antarctic and South Pole.

Bruce R. Wolfe is a principal of Structural Consultants, Inc. (SCI), a Denver, Colorado structural engineering firm. He received a Bachelor of Science degree in civil engineering from Purdue University in 1971. He has also taken post graduate work at Purdue University and the University of Colorado in structural engineering.

Wolfe's work experience is unique as he has worked in construction in the field in high school and college, as an engineer for two precast concrete manufacturers, a project engineer for Petry C.M. and more recently as an engineer and project manager for two Denver consulting firms. He has been with SCI since 1979.

Bruce is the current Chair of the SEAC/RMSCA Steel Liaison Committee, a collaborative effort for the Structural Engineers Association of Colorado and the Rocky Mountain Steel Construction Association with members from the areas of structural engineering, steel fabrication, steel detailing, and steel erection.

He co-authored an article for the PCI Journal title "Design – Construction of United Bank Tower". He is a licensed professional engineer in several states, has served on the Board of Directors for SEAC, and is a member of AISC and PCI.

ABSTRACT

One of the most time consuming and problematic issues with structural steel buildings is accounting for the various exterior wall systems. This paper will discuss and present practical solutions for interfacing the structural steel frame system with different exterior wall systems. Engineering, serviceability, constructability, and economic considerations will be discussed including recommendations for deflection limits for interfacing with metal stud, precast concrete, and curtain wall systems.

SEAC

STEEL BUILDING - EXTERIOR WALL INTERFACE ISSUES

JANUARY 2005

STRUCTURAL ENGINEERS ASSOCIATAION OF COLORADO
ROCKY MOUNTAIN STEEL CONSTRUCTION ASSOCIATION

STEEL LIAISON COMMITTEE

Jeff Janakus, P.E. – Puma Steel; Bruce Wolfe, P.E. – Structural Consultants, Inc.; Tom Skinner, P.E. – JVA Consulting Engineers, Inc.; Rob Leberer, P.E. – Anderson & Hastings Consulting Engineers, Inc.; Jack Petersen, P.E. – Martin/Martin, Inc.; Dave Schroeder, P.E. – Martin/Martin, Inc.; Bill Zimmerman – Zimkor Industries, Inc.; Richard Huddleston – Zimkor Industries, Inc.; Maynard Trostel – Puma Steel; Rex Lewis – Puma Steel; John Stodola – Derr and Gruenewald Construction Co.; Rick Pelletier – SNS Iron Works, Inc.;
and Dave Henley, P.E. – Vulcraft.

Introduction

The Steel Liaison Committee of SEAC/RMSCA has developed the following document to assist Architects, Consulting Structural Engineers, and Contractors in the design/detailing and budgeting/bidding of perimeter slab/deck edge conditions for steel buildings. The information included herein addresses the issues associated with the interface of the perimeter slab/deck edge with various types of exterior wall systems for commercial architectural building projects in the Colorado-Rocky Mountain Region. This document attempts to incorporate the compiled requirements/desires of Architects, Consulting Structural Engineers, General Contractors, Steel Fabricators, Steel Erectors, and Exterior Wall Suppliers and associated construction tolerances for the steel frame and exterior wall systems in a coordinated and coherent manner.

The committee members represent firms from the disciplines of Consulting Structural Engineering, Steel Fabrication, Steel Erection, Steel Detailing, and Steel Joist/Metal Deck Manufacturing. Further input for this project was provided by members who represent firms from the disciplines of Architecture, General Contracting and Metal Stud, Drywall, Precast Concrete, Curtainwall/Storefront, and Brick/CMU/Stone Subcontracting and Engineering.

To this committee’s knowledge, prior to the information contained herein, sufficient information addressing the design/detailing issues pertaining to the interaction between steel building perimeter slab/deck edges and exterior wall system conditions has not been available from a single-source. Thus, Architects and Consulting Structural Engineers have been forced to gather information from multiple sources and/or past experience to complete the design/detailing necessary for construction of projects. These sources can often be hard to find and adequate past experience may not be available, which can lead to details that inadequately define the slab/deck and exterior wall system interface. Subsequently, Contractors have been forced to use judgment and make assumptions based on past experience to complete the information required to budget/bid and then construct the type of slab/deck edge required for the exterior wall system used on the project. Since these past experiences can be varied, ultimately arguments between Owners, Architects, Consulting Structural Engineers, General Contractors, and Steel Subcontractors about what was shown on the Construction Drawings and Project Specifications versus what has been budgeted/priced and what is required for construction of the project occur. Further, if sufficient coordination during the design and/or detailing process between the steel frame and exterior wall system configuration/tolerances is not completed, field modification of the steel frame (cutting and relocating of members) and/or exterior wall system (cutting/modifying of members) may be required, resulting in schedule delays and cost overruns. This document attempts to reduce the amount of confusion associated with this issue.

<u>Note: this document only addresses steel framed buildings with architectural metal stud, precast concrete, curtainwall/storefront, or brick/masonry/stone exterior wall systems – this document does not address structural precast concrete or structural masonry bearing wall buildings with steel floor and/or roof framing. Proprietary steel slab/deck edge framing and exterior wall systems are not addressed by this document. Further, this document is only intended for procedures and conditions typically practiced and experienced by the local design and construction industry for the Colorado-Rocky Mountain Region.</u>

<u>Note: this document is intended to be used as merely a guide and shall not be used in situations where information contained herein is in or believed to be in conflict with governing building codes, material design and/or construction codes, the fundamental principals of structural engineering, design and/or construction standard of practice, and/or unique circumstances that pertain to a particular project type and/or location.</u>

The committee welcomes comments and suggestions on how to improve this document – please forward such correspondence to the Structural Engineers Association of Colorado – Steel Liaison Committee.

<u>Abbreviations</u>

The following abbreviations are used from hereon in this document:

- **“AOR”** for “Architect”
- **“EOR”** for “Consulting Structural Engineer”
- **“Design Documents”** for “Construction Drawings and Project Specifications”
- **“steel edge member”** for “perimeter floor/roof, slab/deck, edge steel angle/bent plate”
- **“HDAS”** for “headed anchor studs”
- **“DAS”** for “deformed anchor studs”

<u>Items of Note</u>

- **STEEL FRAME - WALL SYSTEM “GAP”**: It is important for the AOR and EOR to specify on the Design Documents an appropriate “gap” (dgap) between the steel frame and the exterior wall system. In most instances, many of the problems encountered in exterior wall system to steel frame construction can be eliminated or greatly reduced if this issue is adequately addressed. The “gap” (dgap) however, should not be specified as too large as this causes problems with design of the connection between the steel frame and exterior wall system, creates inflated costs for fire-safing, creates “openings” in the floor between the outside face of the steel edge member and the inside face of the exterior wall system that are difficult to fill with common flooring systems, and decreases the usable versus total square footage ratio for the building. A common specified “gap” is 3/4” or 1/2” for most exterior wall systems. <u>Note: the “gap” (dgap) values specified in this document must be</u>

used in conjunction with the “overlap” (do) values specified in this document – see boxed note in FIGURES 1.1 and 1.3b.

- **STEEL FRAME “OVERLAP”**: It is important for the EOR to specify on the Design Documents sufficient “overlap” (do) of the steel edge member to spandrel beam/girder to allow for adequate field adjustment of the steel edge member to absorb steel column starting point and plumb tolerances. Note: the “overlap” (do) values specified in this document must be used to allow correct use of the “gap” (dgap) values specified in this document – see boxed note in FIGURES 1.1 and 1.3b.
- **DESIGN SCOPE**: It is important for Architects and Consulting Structural Engineers to specify on the Design Documents who is responsible for what aspect of the design of the connections from the exterior wall system to the steel frame.
- **CONSTRUCTION SCOPE**: It is important for General Contractors to fully coordinate who is responsible for what components of the connections from the exterior wall system to the steel frame prior to commencement of detailing.
- **EXPERIENCED INPUT**: The Architect, Consulting Structural Engineer, and General Contractor are encouraged to seek guidance/input from reputable/experienced Steel Fabricators/Erectors and Exterior Wall Suppliers during preparation of the Construction Drawings and Project Specifications, so that efficiency and cost effectiveness can be incorporated.

1 – Steel Framing Slab/Deck Edge Concerns

- Assumptions:
  - Ø The Steel Erector connects the spandrel beam/girder ends to columns/girders and then snaps a straight line along the beam for placement of the steel edge member. Prior to installation of the exterior wall system, the building is plumbed within AISC tolerances.
  - Ø The Steel Fabricator fabricates the steel edge member as and associated kickers, brackets, etc. (as required) in the shop and delivers these to the field loose (not connected to the spandrel beams/girders or columns).
  - Ø The Steel Erector welds/bolts the steel edge member and associated kickers, brackets, etc. (as required) to the spandrel beams/girders in the field.

- Tolerances:
  - Ø A graphic depiction of the steel tolerances issues associated with slab/deck edge construction is given in FIGURES 1.1 and 1.2.
  - Ø A graphic depiction of the tolerances for steel framed buildings and charts specifying steel frame tolerances vs. height of building are given in FIGURES 1.3a, 1.3b, and 1.3c.
  - Ø The tolerances defined herein are for a 40 ft bay with a spandrel beam/girder flange width of 6" or greater – this governs the edge spandrel beam/girder tolerance ('de') that is calculated: (1/8)*(40 ft)/(10) = 0.5".
  - Ø Total Steel Frame Tolerance: The tolerances specified herein for the steel slab/deck edge construction are cumulative from mill and erection procedures. Mill and erection tolerances are cumulative because the Steel Erector has no method of correcting them in the field. Fabrication tolerances however, are not typically cumulative with erection tolerances because the Steel Erector typically can adjust for them during the erection of the building. The tolerances herein have been taken from current AISC documents. Typically, project specifications reference these documents and/or define specific tolerances required for the project.
  - Ø Steel Mill Tolerances: Mill tolerances account for imperfections in the cross-sectional shape/dimensions, straightness, sweep, camber, etc. of steel members incurred during the milling process.
  - Ø Steel Fabrication Tolerances: Fabrication tolerances account for imprecisions and inaccuracies in steel members incurred during the cutting, drilling, welding, cambering, etc. of steel members in the shop.
  - Ø Erection Tolerances: Erection tolerances account for variations in the erected location of column-to-beam intersection points and the alignment of members from theoretical locations of column-to-beam intersection points and member alignments defined in the Design Documents.
  - Ø The steel tolerances dictate how much the steel frame can be "out" from the theoretical location. This is used along with the exterior wall system tolerances which dictate how much the exterior wall system can be "in" from the theoretical position to provide the amount of "gap" (dgap) required between the steel edge member and the inside face of the exterior wall system. Note: the "gap" (dgap) values specified in this document must be used in conjunction with the "overlap" (do) values specified in this document – see boxed note in FIGURES 1.1 and 1.3b.
  - Ø The steel tolerances dictate how much the steel frame can be "in" from the theoretical location. This is used along with the exterior wall system tolerances which dictate how much the exterior wall system can be "out" from the theoretical position to define the amount of "eccentricity" required for design of the steel and exterior wall system framing members. Note: the "gap" (dgap) values specified in this document must be used in conjunction with the "overlap" (do) values specified in this document – see boxed note in FIGURES 1.1 and 1.3b.

- Construction:
  - Ø A graphic depiction of the steel fabrication and erection construction/fit-up/sequencing issues associated with slab/deck edge construction are given in FIGURES 1.4-1 and 1.4-2. Note that the

relative efficiency of fabrication and/or erection decreases from FIGURE 1.4-1a to FIGURE 1.4-1d and from FIGURE 1.4-2a to FIGURE 1.4-2d.

- Ø Performing a task in the shop is more cost effective than performing the same task in the field – shop welding is more efficient than field welding. Typically, shop welding and/or bolting and field bolting is desired whenever possible.
- Ø Squared ends of members are more efficient and cost effective for fabrication than tapered, clipped, rounded, etc. ends of members.
- Ø Connection of members to one surface is more efficient and cost effective for fabrication and erection than to two or more surfaces.
- Ø Down (from the top) or side (from the side) welds are more efficient and cost effective for fabrication and erection than overhead (from underneath) welds.
- Ø A spandrel beam/girder with adequate flange width must be provided to allow sufficient connection of the HDAS and steel edge member to the spandrel beam/girder. AISC provides minimum required flange widths to allow for HDAS to be offset from the beam/girder web.
- Ø Allowance of customized (welded or bolted) connections for the Steel Fabricator and Erector are more efficient and cost effective than dictating the method – each Steel Fabricator and Erector has their own desired connections methods that vary from company to company.
- Ø Angles are easier to work with in fabrication and erection more cost effective than bent plates. Note that plates above 1/4" can have a potential for cracking during the bending process – the thicker the plate, the more potential for cracking.
- Ø Gage metal (thinner than 3/16") is easier to work with in fabrication and erection and more cost effective than plate (3/16" thick or greater). Gage metal however, is typically not suitable for conditions in which it must transfer the exterior wall weight to the steel frame.
- Ø HDAS are more cost effective to purchase and install than DAS. Shop installation of HDAS and DAS is more cost effective than field installation. Field installation of DAS can add significant cost.
- Ø No camber for dead load of the perimeter spandrel beam/girder is preferred (if possible) to avoid problems with the camber not coming out and/or coming out too much during pouring of the slab on metal deck.

- Engineering:
  - Ø A graphic depiction of the steel engineering issues associated with slab/deck edge construction is given in FIGURES 1.5-1 and 1.5-2.
  - Ø It is beneficial for the Design Documents to illustrate the eccentricities that the steel frame is design for and/or the eccentricities that the exterior wall system must be designed for.
  - Ø Typically the concrete slab on metal deck and associated reinforcing is designed to carry the weight of the exterior wall system from the steel edge member to the steel spandrel beam/girder centerline. The steel edge member is typically designed to carry the wet weight of the concrete slab on metal deck to the steel spandrel beam/girder centerline.
  - Ø Eliminating extended slab edges by moving column lines outward can greatly reduce the cost of construction of the steel frame.
  - Ø The Design Documents should clearly define which pieces of steel (i.e. beams/girder, columns, braced frame members, kickers, connection plates, etc.) require fireproofing.

- Slab/Deck Closure vs. Exterior Wall Support:
  - Ø For situations where the edge of slab/deck is not required to carry the weight of the exterior wall system, lighter material such as gage metal in lieu of plate can be specified for the steel edge member.

- In some scenarios, the edge of the slab/deck may serve as only a pour stop for the concrete and a separate steel member may be provided to support the exterior wall system from steel column to steel column. In such scenarios, lighter material such as gage metal in lieu of plate can be specified for the steel edge member.

- Fire-safing:
  - The amount and locations of fire-safing required depends on the fire rating of the building and the requirements of the governing jurisdiction.
  - The cost of fire-safing can be as high as 50% of the cost of the insulation required for the building if excessive gaps between the steel edge member and the exterior wall system are specified. The cost of providing fire-safing between the steel edge member and the exterior wall system can be significant - the cost of fire-safing is proportional to the size of the gap between the steel edge member and the exterior wall system.
  - General Contractors prefer to use fiberglass insulation (if possible) in between the outside face of the steel edge member and the inside face of the exterior wall system.
  - Typically the insulation subcontractor who is contracted with the exterior wall subcontractor supplies and installs the fire-safing.

2 – Metal Stud Exterior Wall System Issues

- Assumptions:
  - Ø <u>The steel structure is fully connected and plumbed with sufficient temporary bracing or the primary lateral force resisting system in place.</u>
  - Ø <u>The Metal Stud Supplier connects their product to applicable perimeter steel frame that has been erected within tolerances indicated in FIGURES 1.1 to 1.3.</u>
  - Ø <u>The Metal Stud Supplier attaches directly to the steel edge member via welds or screws/anchors.</u>

- Tolerances:
  - Ø A graphic depiction of metal stud wall system tolerance issues is given in FIGURES 2.1a and 2.1b.
  - Ø Drywall tolerances from floor to floor are typically 1/8" maximum in or out. Typically, a 3/4" gap works well for buildings of 3 stories or less.
  - Ø Metal stud wall systems are essentially required to be constructed vertically plumb to achieve the required drywall tolerances.

- Construction:
  - Ø A graphic depiction of metal stud wall system construction/fit-up/sequencing issues are given in FIGURES 2.2-1, 2.2-2, and 2.2-3.
  - Ø Tolerances for steel members are less restrictive than for metal studs. This must be taken into consideration when locating steel members within metal stud wall systems (i.e. for header, jamb, lintel, lateral support, etc. support) – a 6" steel member does not typically fit well within a 6" metal stud wall system.
  - Ø Using heavier metal studs (within reason) and eliminating the number of pieces (i.e. kickers, stiffeners, etc.) is typically more efficient and cost effective.
  - Ø Avoiding intermittent kickers and connections between beams (if possible) is efficient and cost effective.
  - Ø Using multiple metal studs (within reason) and avoiding mixing in steel shapes (i.e. angles, channels, tubes, etc.) within the metal stud wall system is more efficient and cost effective.
  - Ø Metal Stud Suppliers typically have little problem with installation if a sufficient gap is provided between the steel edge member and the inside face of the metal stud system. Connection clips are required to achieve the connection and allow for adjustment during construction of the metal stud wall system.
  - Ø Metal Stud Suppliers virtually always have problems when a gap between the steel edge member and the inside face of metal stud is not provided – direct connection from the metal studs to the steel edge member are required. Only on rare occasions has this arrangement successfully been used on buildings no taller than 1-story (connections to steel edge members at roof framing). In some instances, the Metal Stud Supplier may clamp the steel edge member in place and then the Steel Erector comes back and completes the weld to the spandrel beam/girder. This arrangement is not preferred by Metal Stud Suppliers or Steel Erectors due to the inconvenience of stopping and starting of work while the other trade is performing their work and is thus, not recommended.
  - Ø If a procedure whereby the Metal Stud Supplier is to design the metal stud wall system based on design parameters given by the EOR, additional time is required during the bidding process to account for this.
  - Ø Depending on the governing jurisdiction, fire-safing may be required from the outside face of the steel edge member to the inside face of the metal stud wall system or to the outside face of the metal stud wall system.
  - Ø CASE A – Balloon Framing (metal studs run by structure) – STICK BULT SYSTEMS (metal studs are individually installed to the structure in the field):

    - Metal Stud Suppliers prefer to attach to the vertical face of the steel edge member – member of at least 1/4" in thickness is desired.
    - Typically, connection clips that deliver the wall weight to the face of the steel edge member or slip vertically are provided. Installed costs for metal stud to steel edge member connection clips are typically in the $12 to $15 per clip range (2004 prices).
    - The Design Documents should clearly specify the requirement for connection clips so that there is no question of intent during budget pricing or bidding.
    - The capability for allowing vertical slip in the metal stud wall system at some location between each floor is required to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
  - CASE A – Balloon Framing – PANELIZED SYSTEMS (metal studs are installed to the structure as panels in the field)
    - Metal Stud Suppliers prefer to attach to an embed plate placed on top of the slab at the perimeter steel frame – the embed plate should be at least 1/4" thick and 6" wide – a continuous embed plate in lieu of intermittent embed plates is desired. Often the embed plate is required to be recessed into the slab so that it can be hidden under the flooring system.
    - Typically, a continuous strong-back member (i.e. angle, wide flange, tube, etc.) that connects the metal stud wall to the embed plate is provided.
    - The capability for allowing vertical slip in the metal stud wall system between panels is required to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
  - CASE B – Infill Framing (metal studs run in between structural levels):
    - Metal Stud Suppliers prefer to attach to the top of slab and bottom of steel edge member with screws/anchors.
    - The capability for allowing vertical slip in the metal stud wall system at the under side of the steel edge member is required to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
  - CASE C – Strip Panels (metal stud panels hung from the slab edge and kicked back to steel frame – for strip window systems) – CLIPPED CONNECTIONS:
    - Metal Stud Suppliers prefer to attach to the vertical face of the steel edge member – member of at least 1/4" in thickness is desired.
    - Typically, connection clips that deliver the wall weight to the face of the steel edge member or slip vertically are provided. Installed costs for metal stud to steel edge member connection clips are typically in the $12 to $15 per clip range (2004 pricing).
    - The Design Documents should clearly specify the requirement for connection clips so that there is no question of intent during budget pricing or bidding.
    - The capability for allowing vertical slip in the metal stud wall system at the top of the strip window is required to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
  - CASE C – Strip Panels – EMBED PLATE CONNECTIONS:
    - Metal Stud Suppliers prefer to attach to an embed plate placed on top of the slab at the perimeter steel frame – the embed plate should be at least 1/4" thick and 6" wide – a continuous embed plate in lieu of intermittent embed plates is desired.
    - Typically, a continuous strong-back member (i.e. angle, wide flange, tube, etc.) that connects the metal stud wall to the embed plate is provided.
    - The capability for allowing vertical slip in the metal stud wall system at the top of the strip window is required to ensure that the relative deflections of floors do not load/crush the metal stud wall system.

- Engineering:

- A graphic depiction of metal stud wall system engineering issues is given in FIGURES 2.3-1, 2.3-2, and 2.3-3.
- The metal stud system can be either designed by either the EOR or the Metal Stud Supplier. If the Metal Stud Supplier is to design the system, sufficient design criteria (a performance specification) must be specified in the Design Documents by the AOR and/or EOR, i.e.:
  - Stick-built or panelized construction.
  - Metal stud width/depth.
  - Allowable locations of kickers or intermediate horizontal support between floor/roof levels and allowable connection points to structural framing as applicable/allowed.
  - Procedure for installing kickers – i.e. installing after floor/roof dead load is in place to avoid "kicking" out of exterior wall during inducing dead loads as applicable/allowed.
  - Vertical (dead) load of wall.
  - Horizontal (wind) load applied to wall or governing code criteria sufficient to derive loads.
  - Allowable vertical and horizontal deflection criteria.
  - Vertical load eccentricity to be used – i.e. from center of gravity of metal stud wall system to face of steel edge member.

  Since the AOR and EOR are not as familiar with the intricacies of metal stud system construction, this procedure allows for value engineering ideas to be implemented, saving time and costs for the project. To take advantage of these value engineering ideas, a certain level of design control must be given to the Metal Stud Supplier, i.e.:
  - Choice of using heavier metal studs or providing kickers, stiffeners, etc. as applicable/allowed.
  - Choice of metal stud thickness and spacing.
  - Choice of connection types.
- CASE A – Balloon Framing (metal studs run by structure) – STICK BUILT SYSTEMS (metal studs are individually installed to the structure in the field):
  - The connection clips must be designed to deliver the wall weight to the face of the steel edge member or be designed to slip vertically.
  - The Design Documents should clearly specify the requirement for connection clips so that there is no question of intent during budget pricing or bidding.
  - The metal stud system must have capability for allowing vertical slip at some location between each floor to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
- CASE A – Balloon Framing – PANELIZED SYSTEMS (metal studs are installed to the structure as panels in the field):
  - The strong-back member and associated connections that connects the metal stud wall must be designed to deliver the wall weight to the face of the steel edge member.
  - The capability for allowing vertical slip in the metal stud wall system between panels is required to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
- CASE B – Infill Framing (metal studs run in between structural levels):
  - The metal stud system must have capability for allowing vertical slip at the under side of the steel edge member to ensure that the relative deflections of floors do not load/crush the metal stud wall system.
- CASE C – Strip Panels (metal stud panels hung from the slab edge and kicked back to steel frame – for strip window systems) – CLIPPED CONNECTIONS:
  - The connection clips must be designed to deliver the wall weight to the face of the steel edge member or be designed to slip vertically.
  - The Design Documents should clearly specify the requirement for connection clips so that there is no question of intent during budget pricing or bidding.

- § The metal stud system must have capability for allowing vertical slip at the top of the strip window to ensure that the relative deflections of floors do not load/crush the metal stud wall system.

- Ø CASE C – Strip Panels – EMBED PLATE CONNECTIONS:
  - § The strong-back member and associated connections that connects the metal stud wall must be designed to deliver the wall weight to the face of the steel edge member.
  - § The metal stud system must have capability for allowing vertical slip at the top of the strip window to ensure that the relative deflections of floors do not load/crush the metal stud wall system.

<u>3 – Precast Concrete Exterior Wall System Issues</u>

- Assumptions:
  - Ø <u>The steel structure is fully connected and plumbed with sufficient temporary bracing or the primary lateral force resisting system in place.</u>
  - Ø <u>The slab on metal decks are poured and cured prior to installation of precast concrete pieces to the steel frame.</u>
  - Ø <u>The Precast Concrete Supplier connects their product to applicable perimeter steel frame that has been erected within tolerances indicated in FIGURES 3.1 to 1.3.</u>
  - Ø <u>The precast concrete piece to steel frame connections are designed by the EOR or Precast Concrete Supplier.</u>

- Tolerances:
  - Ø A graphic depiction of precast concrete system tolerance issues is given in FIGURES 3.1a and 3.1b.
  - Ø For 1 story buildings where the weight of the precast concrete is carried by the building foundation (precast concrete is not carried by the steel frame), a 3/4" gap is typically provided between the outside face of the steel edge member and the inside face of the precast concrete piece.

- Construction:
  - Ø A graphic depiction of precast concrete system construction/fit-up/sequencing issues is given in FIGURES 3.2-1, 3.2-2a, 3.2-2b, and 3.2-3.
  - Ø The Design Documents should require that a pre-detailing meeting be held with the following parties being present: AOR, EOR, General Contractor, Steel Fabricator/Detailer/Erector, and Precast Concrete Supplier Fabricator/Detailer/Erector. Issues concerning steel/precast concrete plan and elevation locations, cambers and sweeps, tolerances, embed plate/insert/anchor locations and orientations, connection piece scope of works, field weld procedures, erection sequencing, and temporary bracing/shoring should be discussed.
  - Ø The Design Documents should require that the General Contractor distribute all precast concrete system shop drawings to the Steel Fabricator and Steel Erector and all steel shop drawings to the Precast Concrete Supplier for purposes of coordination between trades. Proper and timely coordination between trades can decrease or eliminate costly field fixes.
  - Ø Making embed plates larger (within reason) is more efficient and cost effective than having to provide field fixes for undersized embed plates.
  - Ø Steel edge members should be 1/4" thick minimum.
  - Ø CASE A – For buildings of 1 to 2 stories where the weight of the precast concrete is carried by the building foundation (precast concrete is not carried by the steel frame):
    - § The connection from precast concrete to steel frame should slip vertically and be allowed to rotate with respect to the steel frame.
    - § To allow for slip and rotation, Precast Concrete Suppliers typically prefer to use a slotted insert embedded in the precast concrete pieces. These connections are more efficient and cost effective than loose plate to embed plate connections considering the time/labor that they save.
  - Ø CASE B – For buildings of 2 stories or more where the weight of the precast concrete is carried by the steel frame (precast concrete does not rest on the building foundation) – Precast concrete pieces span from steel column to steel column:
    - § Precast Concrete Suppliers typically prefer this scenario over supporting the precast concrete on the steel spandrel beam/girder.
    - § Precast Concrete Suppliers typically prefer steel brackets (i.e. steel angle, stiffened angles, WT's, wide flanges, tubes, channels, etc.) to bear the precast concrete pieces on. Connection

to the steel column flanges is typically preferred – connection to steel column webs introduces excessive eccentricities to the connection. Typically the order from most cost effective to least cost effective shape to be used for the steel bracket is: angles, channels, stiffened angles, WT's, wide flanges, and tubes.

    - § If it can be coordinated, it is typically preferred by Precast Concrete Suppliers that the steel brackets be placed on the steel columns in the Steel Fabricator's Shop. Otherwise, the steel brackets are typically provided and installed by the Precast Concrete Supplier. It is typically more cost effective for the project as a whole to place the steel brackets on the steel columns in the Steel Fabricator's Shop – shop labor is typically more cost effective than field labor.
    - § Steel brackets should be held down below the theoretical bearing elevation for the precast concrete pieces and shims provided to achieve the correct elevation in the field. This is significantly more cost effective than not allowing a shim gap for "slop" and having the field cut and reattach the steel bracket. Typically a 1" thickness of shims is acceptable.
    - § 2" minimum bearing is required for the precast concrete piece to steel bracket.
    - § Lateral support of the precast concrete to the steel frame is required typically at 4 ft on center.
  - Ø CASE C – For buildings of 2 stories or more where the weight of the precast concrete is carried by the steel frame (precast concrete does not rest on the building foundation) – Precast concrete pieces are carried by the steel spandrel beam/girder:
    - § A continuous strong-back member (i.e. steel angle, tube, wide-flange, etc.) is typically preferred and provided by Precast Concrete Suppliers. Typically the order from most cost effective to least cost effective shape to be used for the continuous strong-back member is: steel angle, tube, and wide-flange.
    - § Two points or more of bearing to the supporting steel frame.
    - § Precast Concrete Suppliers typically prefer the bearing points to be located with the end 1/4 spans of the steel spandrel beam/girder to reduce the adverse effects of concrete slab on metal deck and steel spandrel beam/girder deflections.
    - § Up to 4" of gap from the inside face of the precast concrete piece to the face of the steel edge member can typically be accommodated by the Precast Concrete Supplier. For conditions with gaps greater than 4", the slab edge should be extended outward.
    - § Making embed plates larger (within reason) is more efficient than having to provide field fixes for undersized embed plates.

- Engineering:
  - Ø A graphic depiction of precast concrete system engineering issues is given in FIGURES 3.3-1, 3.3-2a, 3.3-2b, and 3.3-3.
  - Ø The Design Documents should clearly define the responsibility for designing the interfacing connections between the steel and precast concrete framing.
  - Ø CASE A – For buildings of 1 to 2 stories where the weight of the precast concrete is carried by the building foundation (precast concrete is not carried by the steel frame):
    - § The connection from precast concrete to steel frame should be designed and/or specified to slip vertically and be allowed to rotate with respect to the steel frame.
    - § To allow for slip and rotation, Precast Suppliers typically prefer to use a slotted insert embedded in the precast concrete pieces.
  - Ø CASE B – For buildings of 2 stories or more where the weight of the precast concrete is carried by the steel frame (precast concrete does not rest on the building foundation) – Precast concrete pieces span from steel column to steel column:
    - § The Design Documents should clearly specify the locations that it is assumed the precast concrete pieces will be supported from the steel frame.
    - § The Design Documents should clearly specify the load and eccentricity for which the steel brackets were designed for (by the EOR) or must be designed for (by the Precast Supplier).

    - Steel brackets should be held down below the theoretical bearing elevation for the precast concrete pieces and shims provided to achieve the correct elevation in the field.
    - 2” minimum bearing is required for the precast concrete piece to steel bracket.
    - Typically the precast concrete is designed to be laterally supported to the steel frame at 4 ft on center.
- CASE C – For buildings of 2 stories or more where the weight of the precast concrete is carried by the steel frame (precast concrete does not rest on the building foundation) – Precast concrete pieces are carried by the steel spandrel beam/girder:
    - The Design Documents should clearly specify the locations that it is assumed the precast concrete pieces will be supported from the steel frame.
    - The Design Documents should clearly specify the load and eccentricity for which the steel strong-back member is designed for (by the EOR) or must be designed for (by the Precast Supplier).
    - Typically embed plates in the concrete slab on metal deck are designed by the EOR and embed plates in precast concrete are designed by the Precast Concrete Supplier.
    - Up to 4” of gap from the inside face of the precast concrete piece to the face of the steel edge member can typically be accommodated by the Precast Concrete Supplier. For conditions with gaps greater than 4”, the slab edge should be extended outward.

4 – Curtainwall/Storefront Exterior Wall System Issues

- Assumptions:
  - Ø The steel structure is fully connected and plumbed with sufficient temporary bracing or the primary lateral force resisting system in place.
  - Ø The Curtainwall/Storefront Supplier connects their product to applicable perimeter steel frame that has been erected within tolerances indicated in FIGURES 1.1 to 1.3.
  - Ø All connections to steel edge members or embed plates in slabs are by the Exterior Wall Supplier.

- Tolerances:
  - Ø A graphic depiction of curtainwall/storefront system tolerance issues is given in FIGURES 4.1-1a, 4.1-1b, 4.1-2a, and 4.1-2b.
  - Ø Typically, inconsistencies of the steel frame from floor to floor can be accommodated by the Curtainwall/Storefront Supplier if AISC tolerances are met for the steel frame.
  - Ø Typically, Curtainwall/Storefront Suppliers prefer to have a minimum 1" and maximum 4" gap from backside of curtainwall to face of the steel edge member – a 1" gap works well for buildings of 10 stories or less.
  - Ø Curtainwall/storefront systems are required to be installed within 1/16" +/- of vertical plumb in 20 ft vertical – this is non-cumulative from floor to floor.

- Construction:
  - Ø A graphic depiction of curtainwall/storefront construction/fit-up/sequencing issues is given in FIGURES 4.2-1 and 4.2-2.
  - Ø Storefront systems are typically more efficient and cost effective for heights up to 10 ft. Curtainwall systems are typically more efficient and cost effective for heights above 10 ft.
  - Ø Typically, a 14 ft high curtainwall system is more efficient and cost effective than stacking two 7 ft high storefront systems.
  - Ø If storefront systems are placed in front of and are run continuously by supporting steel frame, storefront systems have issues with "thermal break failure" whereby thermal leaks will occur through the wall system. It is recommended that storefront systems span in between supporting steel frame.
  - Ø Rolling over or sweep of the steel edge member is typically not a problem for the Curtainwall/Storefront Supplier.
  - Ø Connections are typically located at each vertical mullion – typically spaced 5 ft on center +/- along the steel edge member.
  - Ø Curtainwall anchors are typically 1/4" to 3/8" diameter (occasionally 1/2" diameter anchors are used for unconventional/heavy systems). The anchors are usually field welded by the Curtainwall Supplier to the mid-height of the vertical leg of the steel edge member.
  - Ø Curtainwall Suppliers prefer to attach to the vertical face of the steel edge member – member of at least 1/4" in thickness is desired. An alternate option is attachment to an embed plate placed on top of the slab at the perimeter steel frame via an angle or plate – the embed plate should be at least 1/4" thick and 6" wide – a continuous embed plate in lieu of intermittent embed plates is desired.

- Engineering:
  - Ø A graphic depiction of curtainwall engineering issues is given in FIGURE 4.3-1 and 4.3-2.
  - Ø Curtainwall/storefront systems typically weigh 10 pounds per square foot.
  - Ø Connections to perimeter steel frame are designed to account for the eccentricity from the curtainwall to the face of the steel edge member.
  - Ø Curtainwall thicknesses are typically 6" for heights up to 13' +/- and 7-1/2" for heights up to 16' +/-.

- Typically maximum lateral deflections due to wind loads of horizontal steel supports for curtainwall/storefront systems should be limited to L/175.
- Typically maximum vertical live load deflections at the perimeter steel frame should be limited to 1/4”.

SPECIAL THANKS FOR THE FOLLOWING CONTRIBUTORS TO THIS EFFORT

Brian Townsend – JHL Constructors, Inc.; John Banks – Sprehe Interior Construction, Inc.; Bob Yelinski – Sprehe Interior Construction, Inc.; Kip Franz – Denver Drywall Company; Gregg Miller – Team Panels; J.D. Schaefer – Stresscon Corporation; Buddy Kirchmar – Rocky Mountain Prestress; John Hanlon – Rocky Mountain Prestress; Joel Watson – Elward; Chris Ciesluk– Elward; Diane Travis – Rocky Mountain Masonry Institute; Mike Schuller, P.E. – Atkinson-Noland & Associates, Inc.; and Irena Kahanek, P.E. – Atkinson-Noland & Associates, Inc.; Mike Murphy – Gerald H. Phipps, Inc., General Contractors; Loren Nelson – Calcon Constructors; David Pfeifer – Anderson Mason Dale Architects; and Dan Horvat – Daniel J Horvat & Associates.

STEEL BUILDING –
EXTERIOR WALL INTERFACE ISSUES

Index of Details and Tables