steelwise

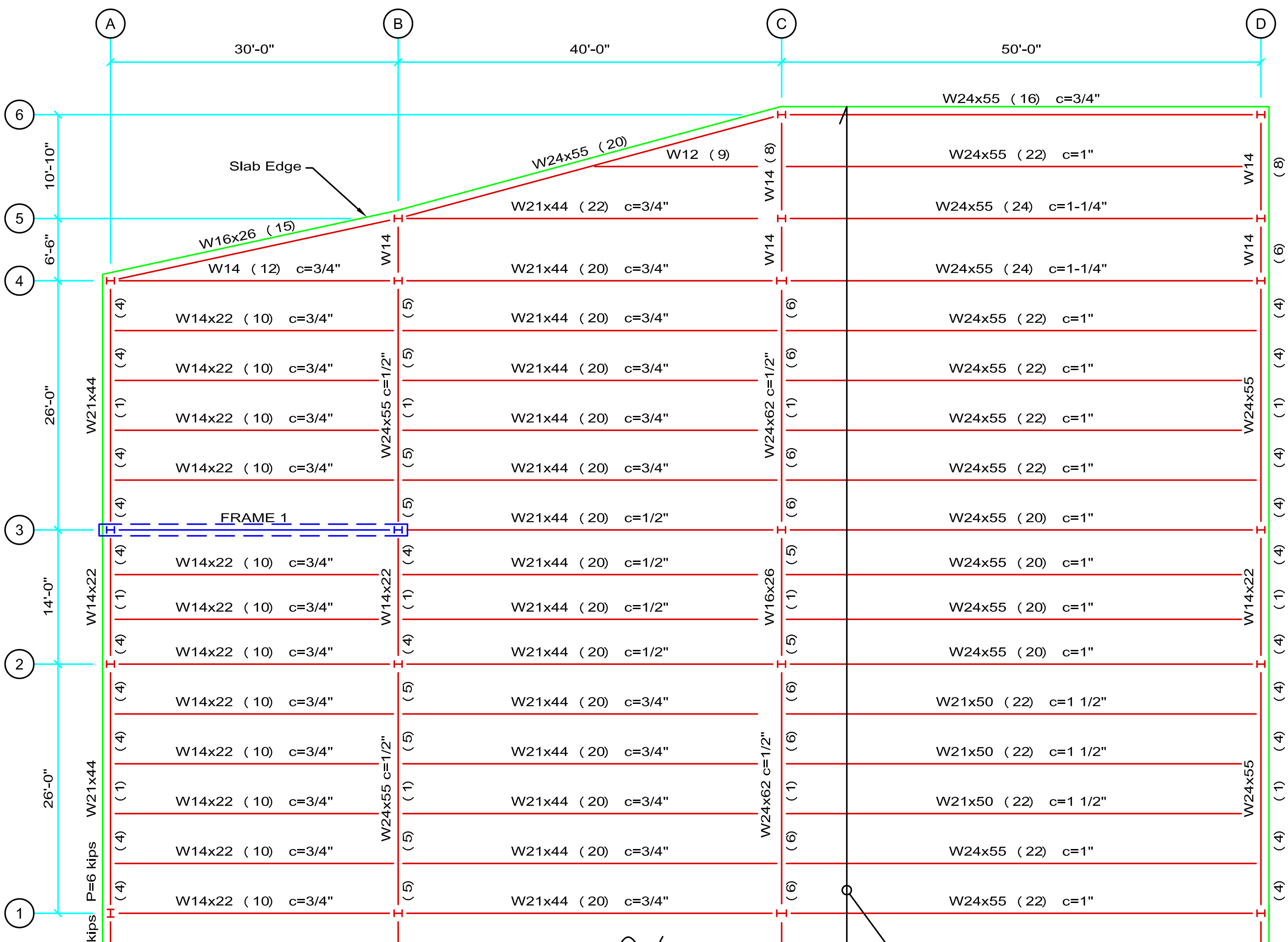
DOES THIS BEAM MAKE MY BUILDING LOOK TOO HEAVY?

An extra pair of trained eyes can often spot opportunities for improvement in a structural framing layout.

BY CARLO LINI

IMAGINE THAT YOU ARE HAVING a conversation with a colleague about a project and you've been asked to give your opinion on a structural framing layout that they've put together. Let's say that the portion of the floor plan provided below, along with the detail and structural steel notes on the following page, is part of the project that's being discussed. What might you do differently?

The ensuing discussion can often make a good design great. To get things rolling, here are 16 suggestions that can help improve almost every design. But first, take a look at the provided floor plan and come up with your own ideas. Then, turn to page 20 to see how well your ideas matched up with ours. (And certainly let us know, via a letter to the editor, what you would have done differently!)



Indicates number of studs.

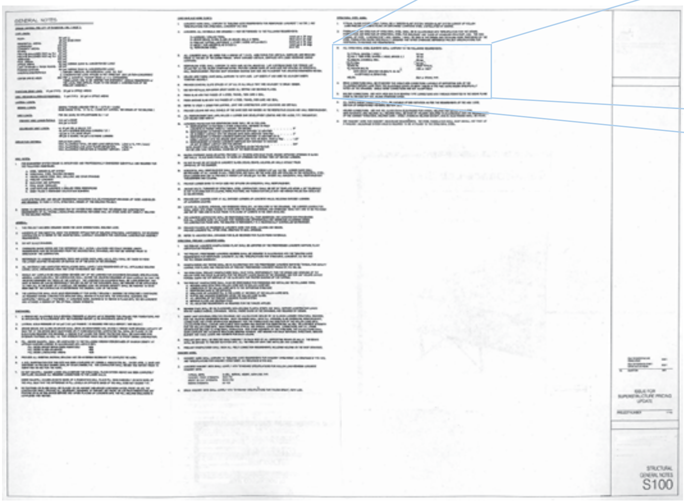
W21×44 (14) c=¾″

Indicates required beam camber.

Note: Top of Floor Slab = 12′-6″

2″ composite metal deck with 4½″ normal weight concrete topping (6½″ total slab thickness)

P= # kips	Axial Design Load (Tension and Compression)
W12	Indicates a W12×14 beam U.N.O.
W14	Indicates a W14×22 beam U.N.O.



3) All structural steel elements shall conform to the following requirements:
Typical shapes: A992, F_y = 50 ksi
Angles, channels, misc.: A36, F_y = 36 ksi
Plates: A572 Grade 50, F_y= 50 ksi

4) Beam connections shall be capable of supporting 50% of the uniform load from the beam tables in the AISC *Manual*.

5) Bolted connections shall use ¾″ ø ASTM A325-N bolts, unless noted otherwise.

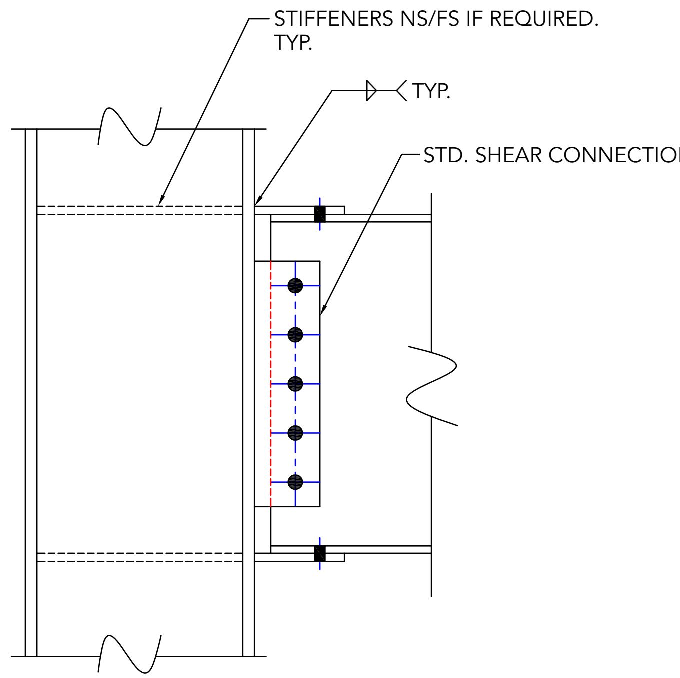
Note that the 16 items highlighted in the following pages address only items that are shown and do not address missing information (e.g., missing dimensions, section cuts, etc.).

1. Eliminate interference at deep beams into shallow girders. Consider connection difficulty versus the cost of increasing a beam to the next size. In this particular case, upsizing the W14×22 to a W16×26 provides some breathing room (see Figure 1) at a cost of about $25 per beam, assuming steel costs $0.45/lb. This will be far less than the labor involved to make a beam fit if the angles foul on the flange or fillet and don't seat freely at the bottom.

2. Eliminate camber in the spandrel beams. AISC Steel Design Guide No. 22, *Façade Attachments of Steel-Framed Buildings*, covers this topic in great detail and sums it up best:

> "…camber in the spandrel beam does not help reduce the size, except for the relatively light… curtain wall system. Any camber in the spandrel beam also places greater tolerance demands on the façade attachments to account for camber tolerances and the uncertainty in the prediction of how much camber will actually come out after loading. Hence, there is wisdom in the common recommendation that camber should be avoided in spandrel beams."

3. Avoid framing a cambered beam into a cambered girder. For cambered beams framing into a girder, the end connections of cambered beams will not be perfectly square. This needs to be accounted for to avoid fit-up issues in the field. Cambering girders combined with cambered beams can increase the probability of fit-up issues. Another reason to avoid cambering girders is that they typically do not deflect as much as beams.



Axial Connection Detail
1) Where Axial Load, P=X kips is posted on Framing Plans, Connection Shall Be Designed for Axial Load Concurrent with Design Shear Load.
2) Use ¾″ ø A325-SC Bolts of 1″ ø A490-SC Bolts for Flange Plate Connection.

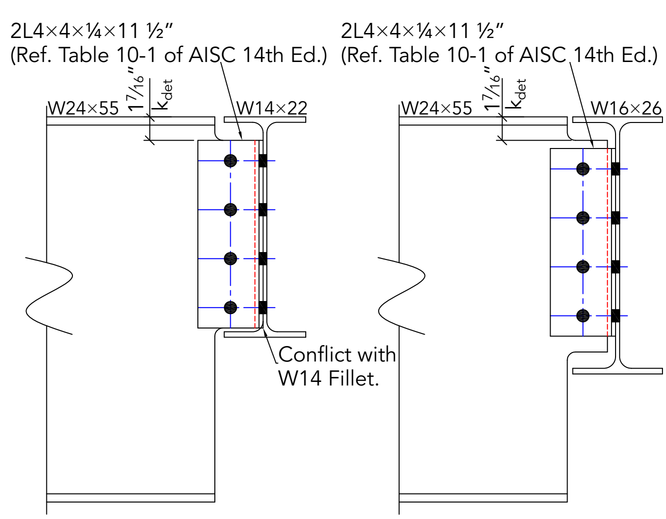


Fig. 1: Deep-beam-to-shallow-girder connection.

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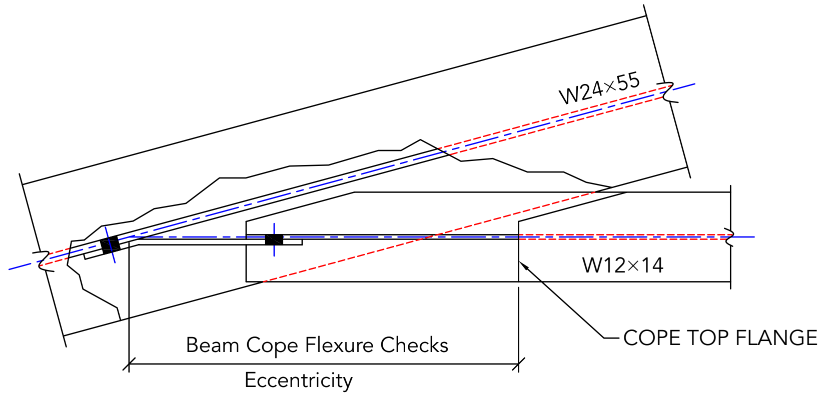
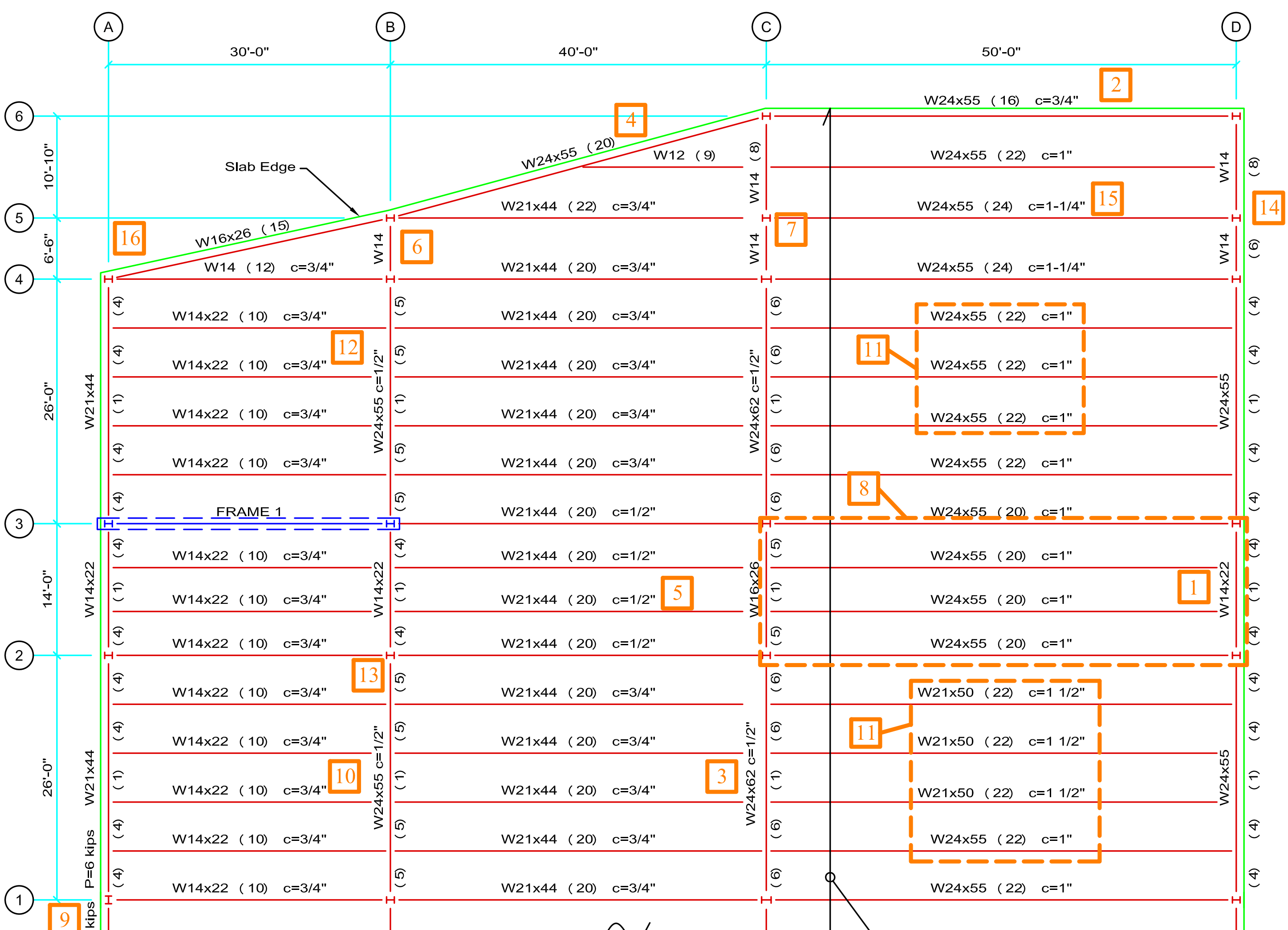
➤ Fig. 2: Difficult skewed beam connection.



4. Change the details at large skews. This is particularly important for shallow beams and beams with large end reactions. This can result in extended shear tabs with thick plates, large welds and extra bolts. Increasing the W12 to a W14 or W16 may help simplify the connection, resulting in a more reasonable shear tab plate thicknesses and weld sizes along with possibly reducing the number of bolts required. If the beam is coped (see Figure 2), reinforcing may be required due to the cope length and shallow beam depth. Increasing the beam depth to avoid beam reinforcement may be a better option.

P= # kips	Axial Design Load (Tension and Compression)
W12	Indicates a W12×14 beam U.N.O.
W14	Indicates a W14×22 beam U.N.O.



W21×44 (14) c=¾" — Indicates number of studs. Indicates required beam camber.

Note: Top of Floor Slab = 12′-6″

2″ composite metal deck with 4½″ normal weight concrete topping (6½″ total slab thickness)

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Another option is to add an additional framing member perpendicular to the direction of the W12 beam. Doing this will simplify the connection for the W12 along with providing a much easier skew connection to the W24×55 (see Figure 3).

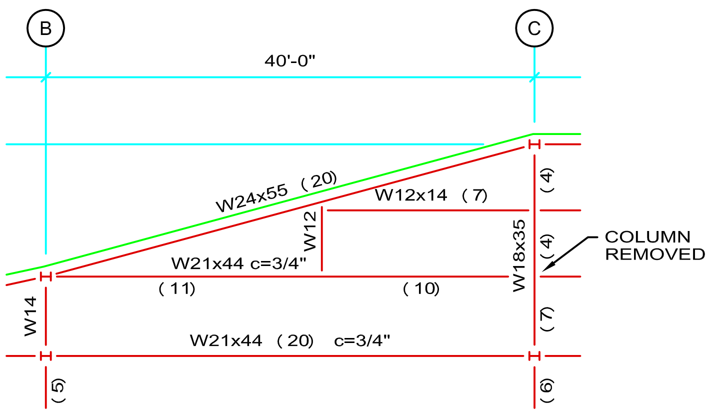

Fig. 3: Skewed beam framing option.

5. Eliminate camber for ordinates less than ¾ in. Small camber requirements usually can be satisfied by natural mill camber. As explained in AISC Steel Design Guide No. 3, *Serviceability Design Considerations for Steel Buildings*:

"It is common practice not to camber beams when the indicated camber is less than ¾ in. The AISC *Code of Stand Practice* provides that if no camber is specified, horizontal members are to be fabricated and erect beams with 'incidental' camber upward. The AISC *Code* also provides that beams received by the fabricator with 75% of the specified camber require no further cambering."

6. Change the connection reactions from UDL to actual values. Using ½ UDL loading (or a similar approach) typically results in less economical connections. At a minimum, provide design loads for cases where beam spans are uncommonly short compared to the beam depth. As an example, consider the W14×22 beam that spans 6 ft, 6 in.; its connections would need to exceed 52 kips to meet the ½ UDL requirements, and the resulting connections will have to be reinforced connections. Designing for the actual reactions in this case will allow standard connections to be used.

7. Can we use longer spans and fewer columns? Steel's high strength-to-weight ratio allows for longer spans compared to other construction materials. Take advantage of this by reducing the number of columns in a project (see Figure 3). Remember that the material cost is approximately 30% of the overall steel structure and the rest is fabrication and erection costs. Reducing the number of columns will help reduce the total number of pieces that need to be fabricated and erected. Plus, fewer columns mean fewer footings. Additionally, the column at D/5 could be removed with similar justification.

8. Let's look at another bay size aspect ratio. Keep an eye on the bay size aspect ratio. If there is enough flexibility within the architectural layout, aim to have a bay length (beam span) that is 1.25 to 1.5 times the width of the bay (girder span). This ratio tends to provide the most economical framing layout.

9. We have a detail that doesn't work for the way the joint was analyzed. The axial connection detail provided on the structural drawings is a fully restrained moment connection, meaning that there is also going to be moment that needs to be transferred. However, the drawings show a joint with shear and axial force only. Either the moment needs to be provided on the framing plans or the axial connection detail needs to be modified. For example, a single-plate shear connection can be used to transfer the shear and axial load (no paddle plates and no moment).

10. Can we increase the beam spacing and have fewer infill beams? When possible, take advantage of the strength of the floor slab and increase the beam spacing. In this case, a 2-in. composite metal deck (20 ga) with 4½-in. normal weight concrete can span unshored up to 9 ft, 0 in. clear and still meet the superimposed loading requirements. Using the strength of the floor slab, two beams can be removed per bay (see Figure 4), which results in a decrease in tonnage of about 1.5 psf for the bay shown, along with fewer pieces to detail, fabricate and erect.

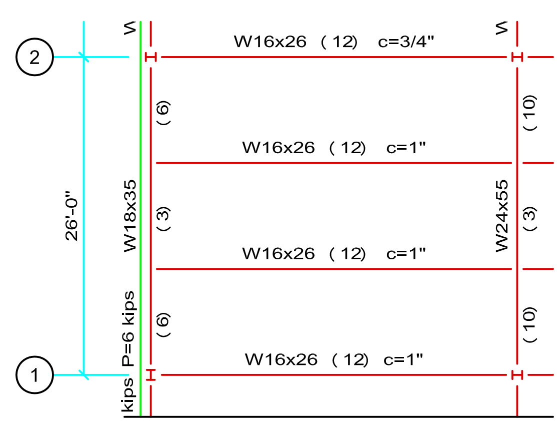

Fig. 4: Increased beam spacing.

11. Can we better group the beam sizes we are using? If the same W24×55 can be used in place of the W21×50, the fabricator can use the same piece mark for this beam instead of having two different piece marks. This helps speed up detailing and fabrication while reducing the possibility of costly errors.

12. Avoid cambering beams with a web thickness less than ¼ in. thick. They may experience web crippling in the camber jig. For this project, the W14×22 beams could be upsized to W16×26 at the approximate cost of $55 per beam. However, this cost may be a wash, since a W16×26 would require no camber.

13. Are all the columns in their best orientation? A column is typically oriented so that girders connect into the column flanges.

14. Can we change the short-span beams and girders to non-composite? Take a look at sizing short beams and girders as non-composite. In some cases the same beams sizes can be used without having to use any studs. The same W14×22 spanning 6 ft, 6 in. and 10 ft, 10 in. can be sized as non-composite saving 14 studs. If the cost of each stud installed is approximated at $3 to $5 per stud, that's a savings of $40 to $70 per beam.

15. Cambering long beams. When cambering beams over 40 ft long, multiple pushes may be required and consistent results are difficult to achieve. It's a good idea to talk this over with a fabricator in deciding what the best option is.

16. Can the details be modified to simplify beam-to-column connections? Try to lay out the floor framing in a way that limits or eliminates complicated connection details. The original layout had a complicated beam-to-column connection at columns A/4 and B/5 (see Figure 5). Although there are several different ways of fabricating the ends of these beams such that the flanges clear each other, some blocking, compound cuts or some combination thereof would be required to allow fit-up. Furthermore, access for providing the welds of the single-plate shear connections to the column flange is tight. Note also the inherent eccentricity induced as a result of where the spandrel beam connection intersects the column flange. This induces a biaxial moment into the column that more than likely was not accounted for in the analysis.

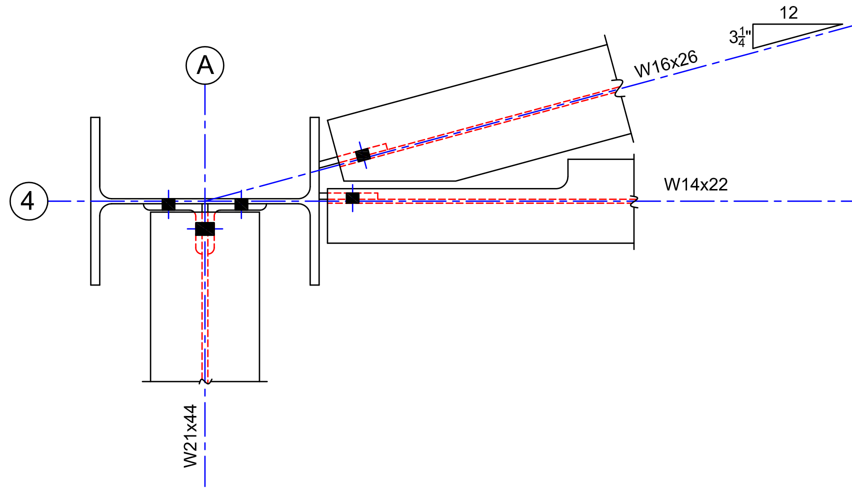

Fig. 5: Complicated beam to column connections.

By changing the orientation of the framing (as shown in Figure 6) and changing the column orientation to line up with the spandrel beams, the complicated connection details are eliminated along with the biaxial moment in the column. From a cost standpoint, this change would have little to no impact on the cost of the steel, but would significantly simplify the design of the connections and reduce the labor involved in prepping the ends of the members.

Giving your advice to a peer (and seeking theirs for your work) is a great way to share knowledge and past experiences with other engineers, and is an opportunity to teach and learn new things. Fresh eyes can see what is invisible, and the discussions will help all to stay sharp on what you already know. Plus, as an added bonus, it will help achieve a better overall design. MSC

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Fig. 6: Modified framing layout.