7.0 Analysis 3: Case Study - Concrete Over-pour on Decks Due to Steel Deflection (Structural Breath)

7.1 Background

A constructability issue that is not unique to this project is the over-pouring of concrete on decks due to steel deflection in order to meet floor flatness (FF) and levelness (FL) specifications.

Some buildings require stricter floor flatness and levelness criteria than others. For example, a warehouse often requires strict guidelines because if a fork-lift is hoisting material in the air, it is crucial that the floor is level so it does not cause the fork-lift to overturn.

JHH NCB requires a moderately strict FF specification of 25 (equal to ¼" over 10'). There is no FL requirement in the specifications. However, the structural drawing notes call for the floors to be poured exactly level, regardless of deflection (see Figure 26 below). The main driving force for this specification is the sophisticated medical equipment throughout the building and the need to roll patient beds around the building.

CP-4 ALL CONCRETE SLABS ON COMPOSITE METAL DECK AND BEAM CONSTRUCTION SHALL BE UNSHORED, UNLESS OTHERWISE NOTED. CONTRACTOR SHALL SUPPLY THE ADDITIONAL CONCRETE REQUIRED TO LEVEL FLOORS DUE TO DEFLECTION INDUCED BY THE WEIGHT OF THE CONCRETE.

Figure 26: Structural Note CP-4 on drawings

7.2 Problem Statement

The concrete subcontractor poured the concrete decks to finish floor elevation, not to deck thickness which is required per note CP-4. The concrete acts as a live load during the pours and on this project caused the steel to deflect up to [redacted] in some cases because no shoring was required.

The concrete contractor was responsible for all over-pour which amounted to 1,200 CY in concrete. This amounted to a substantial amount of money and labor. Another potential problem from this could have been overloading the floor which would have required reinforcing the structure. Finally, the deflection has the potential to impact the coordination of the MEP systems in the ceiling plenum. It is not yet known if that will be a problem on this project because the MEP is not installed in the problematic areas of the building.

7.3 Goal

The goal of this analysis is to examine how the concrete over-pour issue was addressed in the design, bid, and construction phases. This will require the calculation of deflection and over-pour concrete for a typical bay. Finally, a strategy for addressing this problem in each of the phases will be suggested.

7.4 Resources

Turner Construction – Jim Faust
Thornton Tomasetti – Zach Kates (Structural Engineer)
Clark Construction – Joe Salerno
Clark Construction – Lynore Arkin-Yetter
Clark Concrete – Steve Dare
Penn State – Dr. Linda Hanagan

7.5 Analysis

Design Phase

The structural engineer for this project was contacted to discuss how this issue was incorporated in the design. The following is what was learned:

- Predicting deflection under loads is quite difficult because actual loads can vary significantly from design loads
- Only use camber on beams because the loads on their distributed area is predictable
    - Typically camber accounts for 75% of total deflection
- Cannot predict the loads on girders because the distributed area is much larger and unpredictable
    - Do not camber girders
- Typical to require level floors in hospital
- Note CP-4 is not common
- No constructability consulting was done with a contractor
- Engineer allowed 7 PSF for concrete over-pour in construction load design

The typical bay is 28'-8" by 28'-8" with W21x57 (c=0") girders and W16x26 (c=3/4") beams with 3 equal spaces (see Figure 27 below).

Design Loads
Construction Load = 85 PSF (includes 7 PSF for concrete over-pour)
Superimposed Load = 20 PSF
Live Load = 100 PSF

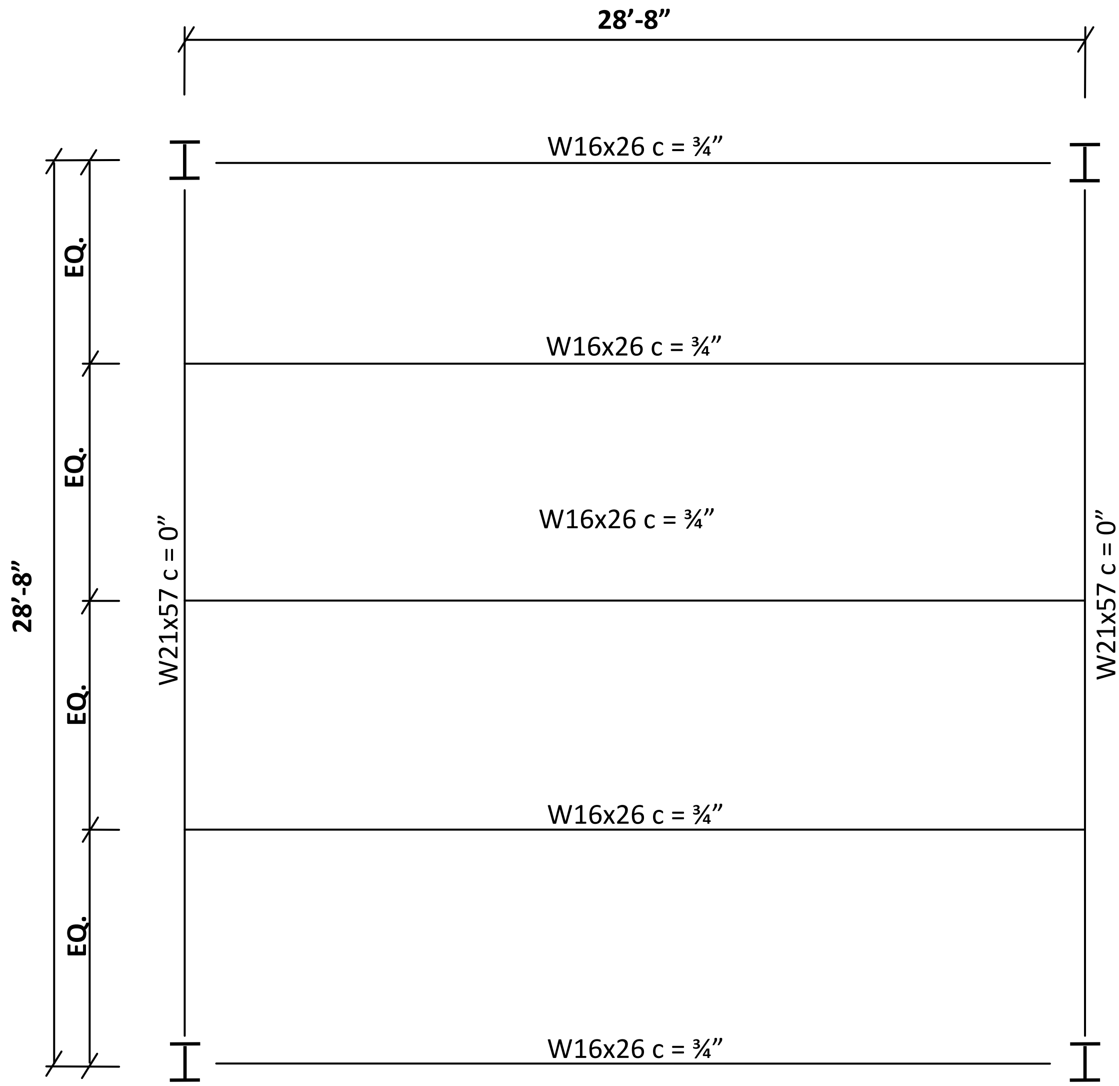
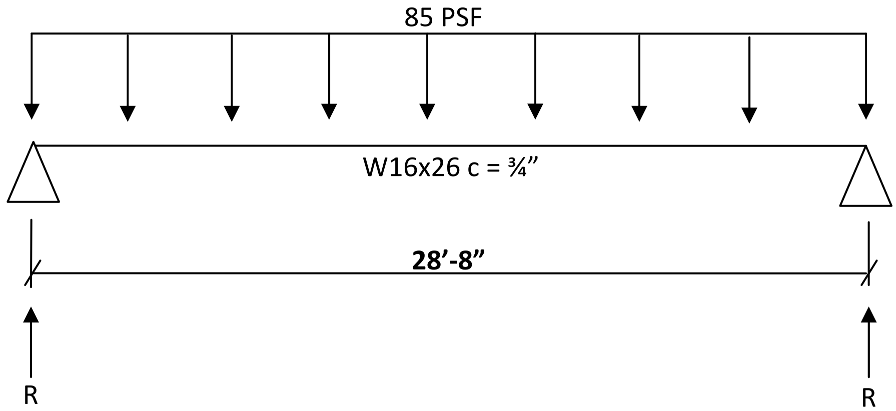


Figure 27: Typical Bay Plan

Typical Beam Deflection – Under Construction Loading

Simply Supported Beam – Uniformly Distributed Load



w = 85 PSF x (28’-8”/3) = 812.2 plf

R = wl/2 = (812.2 plf x 28’-8”)/2 = 23.3 k

Deflection Max (midspan) = $5wl^4/384EI$ = [5 x 812.2 plf x (28.67’ x 12”/’)^4]/(384 x 29x10^6 x 301 in^4 x 12)
= 1.41”

What happens when the beam is up-sized?

Try W18x35 -> I = 510 in^4

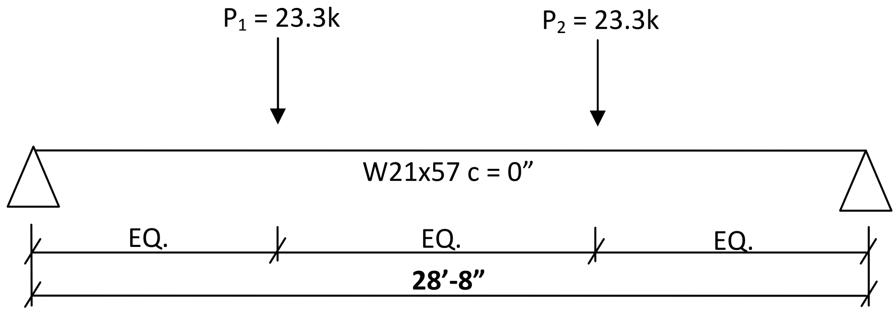
Deflection Max (midspan) = 0.83”

Difference = 0.58”

Since this is the most common steel member in the building it would not be feasible to upsize the beam because it will weight 9 plf more which is the driving force in steel material cost.

Typical Girder Deflection – Under Construction Loading

Simply Supported Girder – 2 Equal Concentrated Loads Symmetrically Placed



$R = P_1 = P_2 = 23.3$ k

Deflection Max (midspan) = $(Pa/24EI) \times (3l^2 - 4a^2)$

$= \{[(23.3 \times 28.667 \times 12)/3] / (24 \times 29{,}000 \times 1{,}170)\} \times [3(28.667 \times 12)^2 - 4(28.667 \times 4)^2]$

= 0.99”

Allowable Deflection = L/360 = 0.96”
Very Close – OK

What happens when the girder is up-sized?

Try W24x55 -> I = 1,350 in^4

Deflection Max (midspan) = 0.85”

Difference = 0.15”

Assuming this member would still meet design shear and moment loads, it would not be worth going to a 3” deeper member because it would reduce the ceiling plenum space by 3”.

Total Volume of Deflection

Beam Deflection = 1.41” – 0.75” (camber) = 0.66” (See Figure 28 below)

Girder Deflection = 0.99” (See Figure 29 on the following page)

Total Deflection in the Center of the Bay = 1.65” (See Figure 30 on the following page)

Assumption: Deflection can be approximated as a triangle

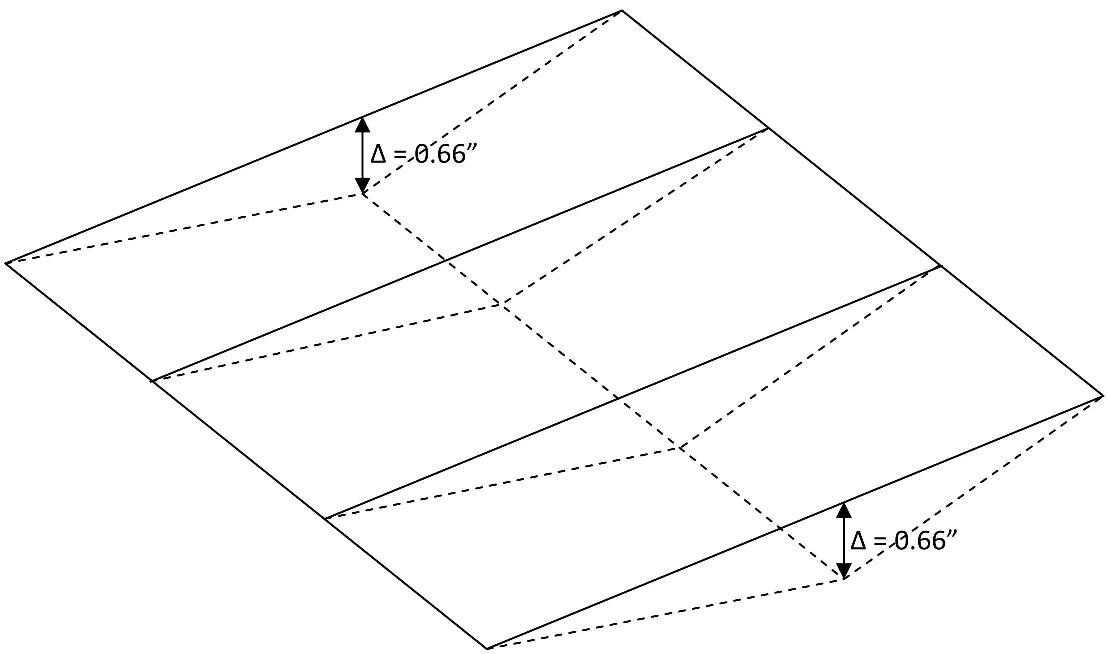


Figure 28: Illustration of Beam Deflection (No Girder Deflection Shown)

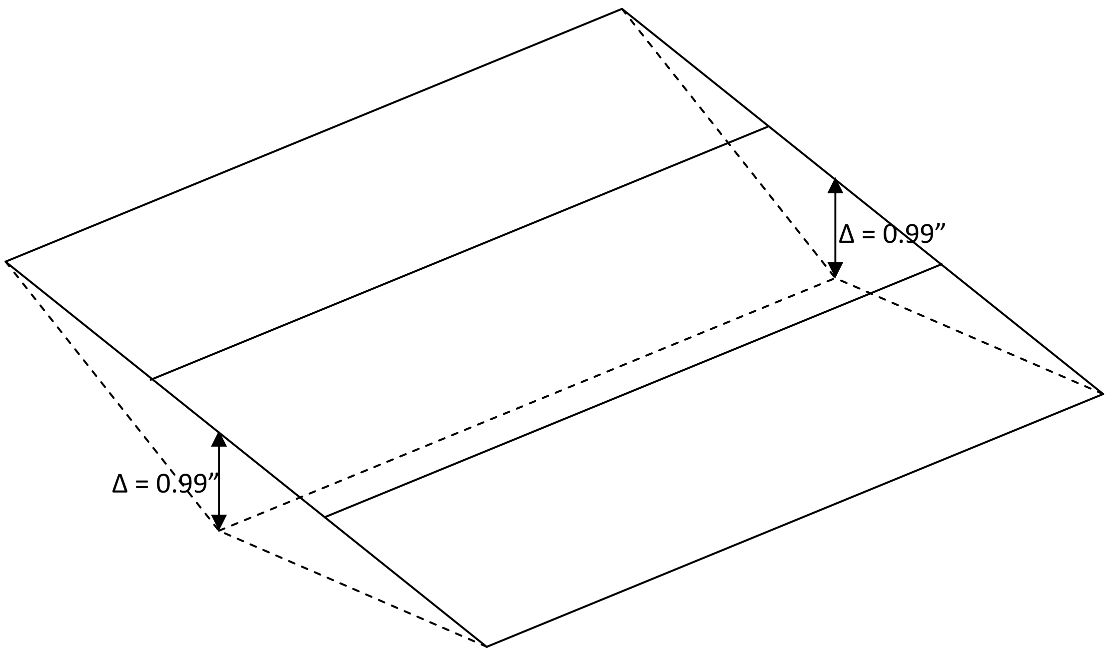


Figure 29: Illustration of Girder Deflection (No Beam Deflection Shown)

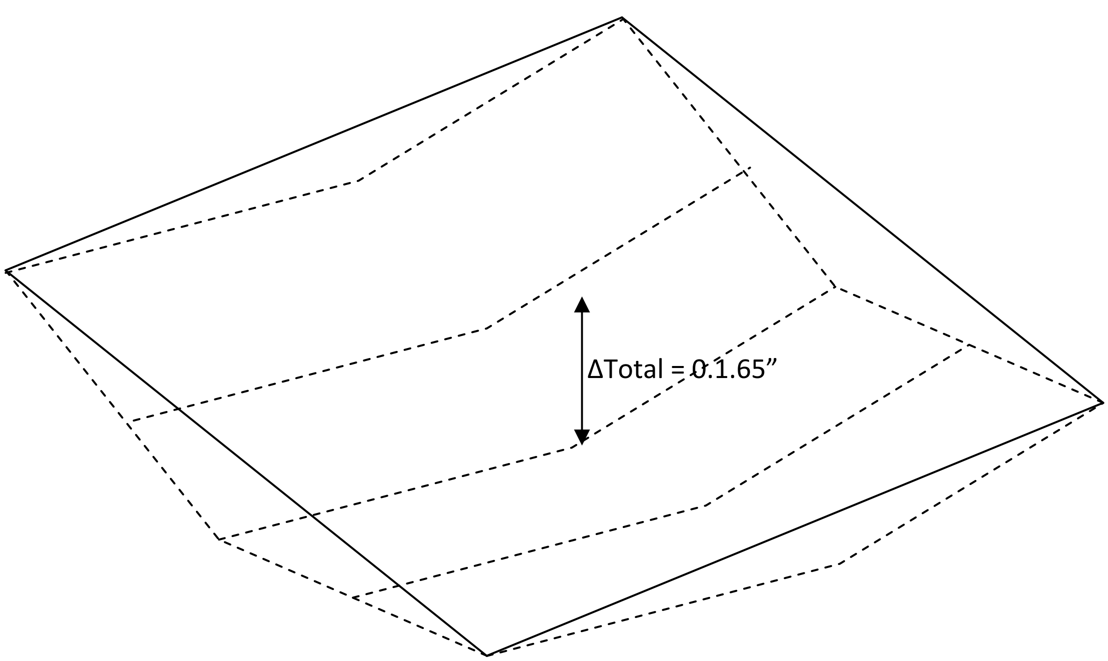


Figure 30: Illustration of Total Deflection (No Center Beams Shown for Visualization)

Volume of Area for Beam Deflection = [(28.667' x 12" x 0.66")/2] x (28.667' x 12") = 39,052 in^3

Volume of Area for Girder Deflection = [(28.667' x 12" x 0.99")/2] x (28.667' x 12") = 58,578 in^3

Total Volume = 39,052 + 58,578 = 97,630 in^3 = 56.5 ft^3 = 2.09 CY

Load of Over-Pour Concrete

150 lb/ft^3 x 56.5ft^3 = 8.47 k of concrete to make floor level

8.47 k / $(28.667')^2$ = 10.3 PSF

Approximate Quantity of Over-pour Concrete

Assume that the total volume of over-pour concrete is spread over the entire area of the bay equally.

56.5ft^3 / $(28.667')^2$ = 0.07' = 7/8"

Total SF of Building = 1.5M

Total Concrete Over-Pour = 1,500,000 SF x 0.07' = 103,128 ft^3 = 3,820 CY

Note: This assumes that every space in the building is a typical bay which is not correct and would likely overestimate the amount of over-pour concrete.

Bid Phase

When Clark/Banks asked Clark Concrete to bid on the project, they pointed out note CP-4. Clark/Banks' lead superintendent was very concerned about the requirement to pour floors level regardless of deflection. He asked them to carry some money for flash patching because he thought it may be a problem.

Clark Concrete never contacted the architect or structural engineer to get an idea of how much they anticipated the floors to deflect. They used historical data and past experiences to estimate the amount of deflection. They assumed 10% extra concrete for deflection and waste in the bid.

Clark Concrete decided to carry an allowance of $100,000 for reshore and flash patching. The shoring would be used for areas where deflection was excessive. The remaining allowance for flash patching would be used for areas that did not meet the levelness criteria.

Construction Phase

Before any of the decks were poured, Clark/Banks held a pre-construction meeting with Clark Concrete. The structural engineer was not asked to attend. Three options were discussed as possible ways to address the problem:

1. Shore the steel to reduce deflection until the concrete cures and can support some tension
2. Pour concrete to thickness by wet-sticking the concrete and coming back later to flash patch areas that were not level
3. Pour concrete to level and pay for the extra concrete

Clark Concrete concluded that option 1 required extensive labor to set-up shores and it was unclear how much creep could be expected after the shores were removed. Option 2 also required extensive labor. It would also mean that they would have to wait until the concrete cured before they could flash patch. By this time other trades may be working on the floor which would have made it difficult to work around them. Clark Concrete finally decided that option 3 was the best because it had the least amount of risk.

Clark Concrete began pouring the decks and kept track of the over-pour concrete by checking the truck tickets. They found that deflection was a significant problem but they believed that they could cover the over-pour concrete with the $100k allowance for reshore and flash patching.

While pouring concrete on the cantilever structure on the south face of the CT, the steel deflection began to become a serious problem. The cantilever structure is a truss system with cross bracing (see Figure 31 on the following page). As the lower levels of the cantilever were poured it became evident that the cantilever was sagging because the steel cross bracing on the floors above were not lining up with their bolt holes.

The connections were slip critical and had slotted bolt holes (see Figure 32 on the following page). A survey later conducted concluded that the steel was set in place by the crane and was not surveyed to make sure the steel was set level. The slotted holes in the connection allowed the steel to sag about ¼”. It was also found that the columns were set ¾” low. This could have been caused by settlement or the base plates may not have been grouted correctly. This all added up to a 1” sag in the cantilever.

The structural engineer required additional gusset plates to be welded on the connections to strengthen the structure. The following floors were surveyed during construction so the steel was set correctly.

Following these findings, Clark/Banks realized that steel deflection was a significant problem that had the potential to impact the MEP coordination. While all of the coordination was being done with 3D BIM, the tolerances were very close. Clark/Banks MEP coordination team decided to allow a 1 ½” buffer for steel deflection. To date they have not encountered any conflicts with MEP and steel deflection.

By the time that all of the concrete decks were poured out, Clark Concrete had poured 1,200 CY of extra concrete due to steel deflection. The extra cost was approximately $100,000 for the over-pour concrete. Clark Concrete decided to use their allowance for reshore and flash patching to cover this instead of submitting a change order to the owner. During construction they had no difficulty meeting levelness and FF requirements so they did not have to use any of the money in this budget.

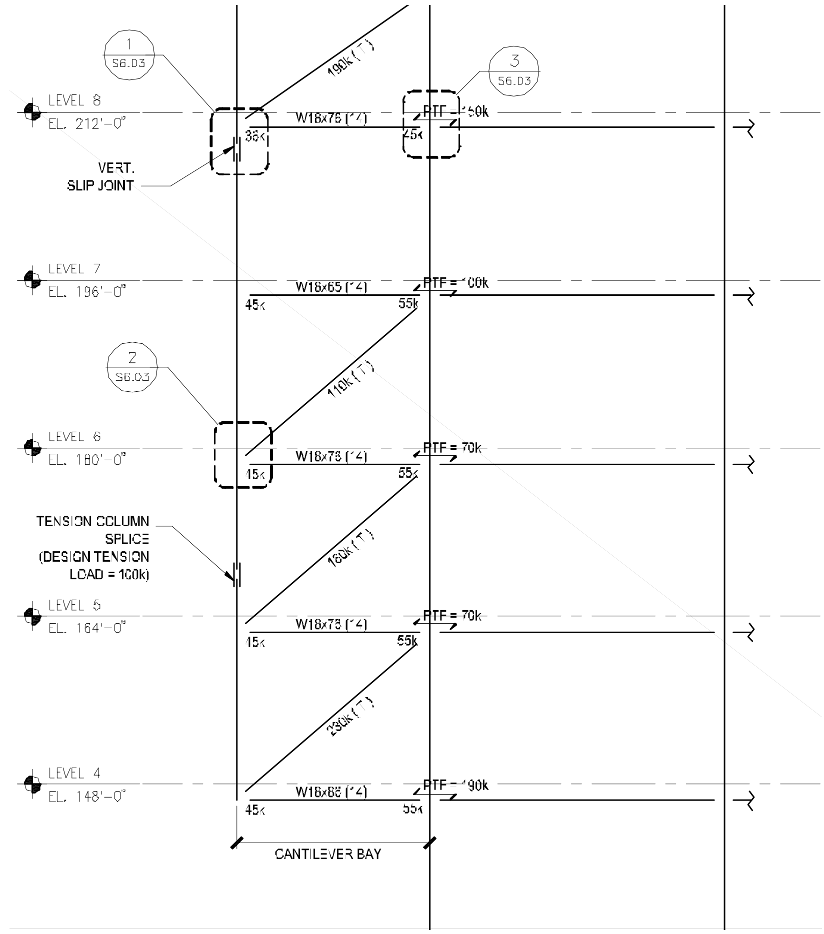


Figure 31: Structural drawing S6.03 depicting the cantilever structure on the south face of the CT

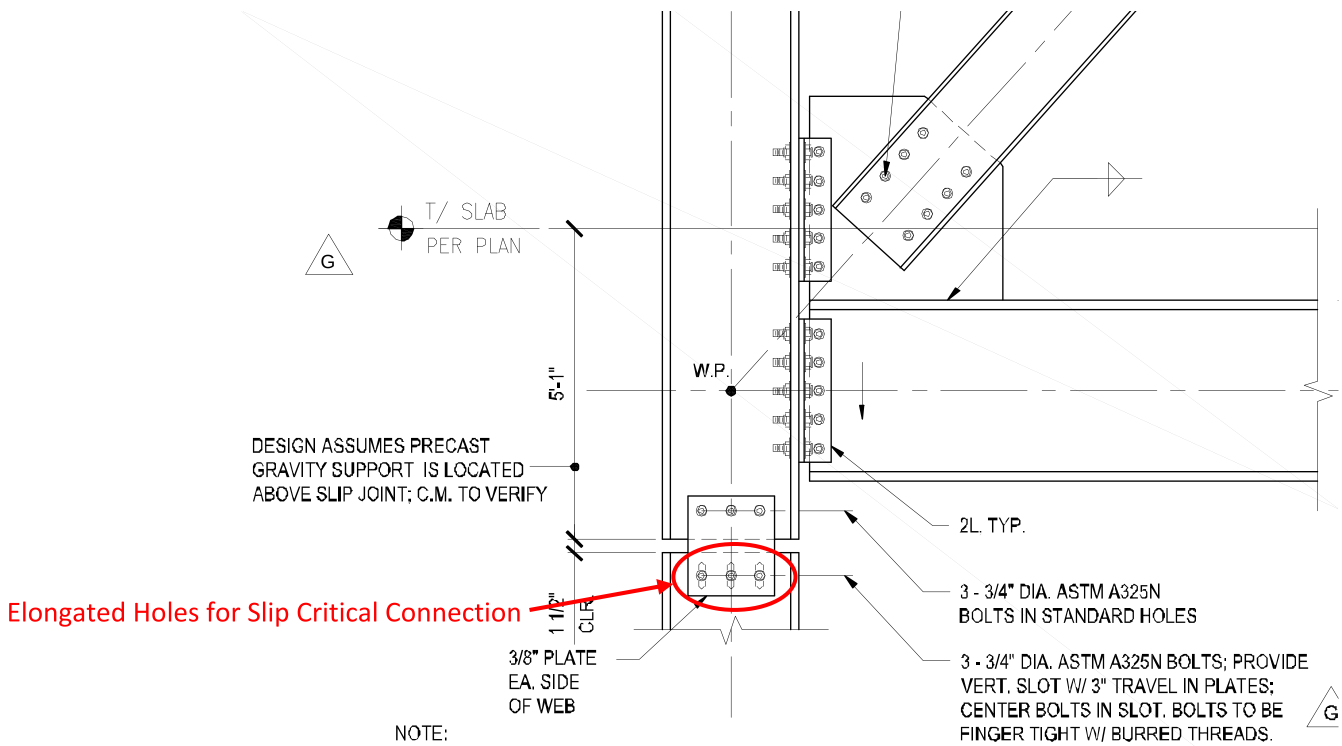


Figure 32: Structural Detail of Slip Critical Connection

After the decks were poured out the structural engineer conducted a survey of the concrete decks. They found that the typical bay deflected on average 1 ½" as predicted by the deflection calculations. The engineer ran the calculations and determined that the structure was capable of supporting the over-pour concrete.

7.6 Conclusion

The problem of over-pour concrete due to steel deflection on metal deck floors is a common problem on construction sites. The degree of significance varies depending on FF, FL, bay size, steel members, and owner requirements. Most projects can get away with just wet sticking the concrete to thickness and still meet FF and FL requirements. The important point of this analysis is to be aware of the requirements and determine if it will a problem on the project in question.

It was clear from the beginning that the JHH project was going to have a steel deflection issue when constructing the decks. By examining what happened on this project, I have identified several areas of improvement. The following is a list of suggestions and recommendations that can be applied to any project.

<u>Design</u>

- Any FF, FL, or notes similar to note CP-4 should be clearly called out in the specifications and drawings.
    - An industry standard for location and format would be most beneficial
- The structural engineer should calculate the *predicted* deflection for a typical bay and should include that in the contract drawings as a guideline, not a specification.
- In buildings that require strict FF and FL requirements such as hospitals and warehouses, the engineer should consult with a contractor for a constructability review.

<u>Bid</u>

- The concrete contractor should request from the A/E the expected deflection for a typical floor.
    - The contractor should create an allowance based on this figure so that any cost under the allowance can be given back to the owner and any cost over can be covered by the owner.

<u>Construction</u>

- The pre-construction meeting should include the structural engineer, steel contractor, flooring contractor, and MEP coordination staff.
- After each floor is poured, a simple survey should be conducted to determine the amount of deflection to check to see if it is as expected.
    - Consult with structural engineer to make sure that the structure is capable of handling extra concrete load.
- The camber in the beams should be checked in the shop and in the field to make sure the correct amount of camber exists.
- An allowance equal to the expected girder deflection (in most cases) should be included in the MEP coordination.

The biggest lesson learned from this case study is that a more integrated team approach from the beginning could have eliminated most of the risk and therefore the cost impact that goes along with this common constructability challenge.