# Open Sectioned Crane Runway Girders with Arbitrary Profile Geometry

## Chapter 5 – Understanding, Misunderstanding

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By natural instinct – or on obvious reasons – many Structural Engineers would stay away as much as they can from being assigned to deal with **Open Sectioned Crane Runway Girders with Unsymmetrical Section or in other words that of Arbitrary Profile Geometry** (**CRG**).

It should be little misunderstanding there as to why making such a favorable-unfavorable choice.

Consider the finer feeling of an individual came across the subject of **<u>unsymmetrical sectioned girders</u>** much as expounded upon in this Series:

> To a great extent to those Lighthearted Structural Engineers who maintain at arm's length from the core of the matter; while on their leisure pursuit, it takes minuscule aspiration for them to shun off the opinions appeared in the preceding Chapters
>
> But then when coming down to *doing the real thing by the real rules in the real time*, they sure need to contend with "a little bit" of challenge prior to even laying the groundwork for good measure, of which such as locating the **shear center for unsymmetrical sections** and calculating the whole slew of associated must-have section properties – and that alone is well qualified as a special topic all by itself

Understandably, to many "pioneers" who had already "gained" indelible experience in tackling comparable challenges further than laying the groundwork and beyond, their tiresome feeling towards handling the subject matter must be exceedingly profound for a fact; so when driven recurrently by such a bittersweet sentiment, it's no surprise that, their top choice between doing it and not doing it would come down to avoiding all **CRG** troubles all together at all cost.

But even at all avoidance of **CRG-themed** challenges at all cost if so chosen, still, we cannot avert facing the reality in the real world; because **unsymmetrical sectioned structures** are not going away any time soon, perhaps never will – that is established with reference to not only **CRG** applications but could also apply to other **non-CRG** works in unavoidable situations.

In spite of rendering our absolute best efforts to **resolve** every instance of engineering-design issue and to **overcome** every **CRG-themed** challenge that came our way, it is not prudent to leave open issues *unresolved* for all (future) time.

Whether we take on those issues willingly or not, the reality is, sooner or later all loose ends must be tended to in due course by someone. What matters the most at this point is who is going to take care of that, by what means, at what cost and by when.

### 5.1 Recognizing the Never-Ending Challenges

Ever since from the way back era for being an essential element of Material-handling- related Mill functions involving overhead cranes, *various configurations* of **CRGs** were made to order, qualified or incorrectly qualified, evaluated, and/or reevaluated suiting explicit purpose as called for in a variety of projects and applications.

On diverse intents, myriad of engineering goals involving **CRGs** were pursued to fulfill countless well-defined project commitments. But then, be whenever the project was over and done with, and, irrespective of whichever engineering approach was adopted for the mission, but in the long run – based on numerous official records came about from so many Mills – albeit with no unkind intent to sell/tell off unfavorable intelligence to those who had never been informed of the real-life story;

> There is always a sporting chance that *some of those girders might not last long enough to fulfill bona fide function suitably as expected by their original outfit*
>
> **A very good sample of very bad example of that is the ever popular cap-channeled girder**

Many present-time Engineers who had immersed so deeply in this same old same-old subject might come to agree with one thing – by everyone's reasonable belief on what *should have been* the case – that is,

> Through more than a century of foregone tried and true "contributions" creditable to so many predecessors, valuable knowledge privy to bettering the engineering treatment to **CRGs** *must have been* accrued aplenty and ready to be shared for everyone's benefit. Wouldn't it?

In spite of what we had hoped for like that, but on various occasions as our Primary Engineering Commonsense advances onward to this day, the baffling question over the uncertainty still holds out;

> Are **CRGs** getting an entitled engineering treatment at all?

Certainly not, and as matter of fact, a number of **much newer breed of CRG-Specific Challenges** could nonetheless emerge out of the blue while fulfilling certain structural assessment/re-assessment missions.

> Some of these **Challenges** we are talking about might be age-old, but could appear novice or give an impression as if still brand new to those who had never "run into" or "gotten out of" the tough situations before
>
> The reality is, most **Challenges** were not new in substance at all; only trouble is, the same **Challenges** were "ignored" too many times over

No matter in a winning or losing position to face whatever the **Challenges**, we should realize there is a much bigger tri-fold setback going on unheeded for too long,

- It can be identified, not all the **CRG-Specific Challenges** we faced through different eras were of exactly the same trait. Challenges can be all different in nature as they are, barring for one thing, all **CRGs** can be led to meet the same fate if the challenges were not dealt with in time properly

  What proven detrimental to the structure that constantly precipitates no-laughing-matter to the Mill Management is, all historical girder failures were of the same old same-old symptom – material fatigue related, if not yet due to material yielding

- Many published manuscripts claimed to be "Design Guide" on Crane Runway Girders had chosen the easy way out, the worst part out of which is by (1) misleading the Readers with indiscriminate or collection of information, (2) providing incomprehensive design examples and (3) meanwhile omitting heaps of vital guidance that are critical for qualifying the adequacy of structures against extended cause of catastrophic failure

- The majority of engineers, although thrived on desk-bind duties, are not adequately trained to recognize the importance hidden in the connection between (1) what omitted in their as-design deliverables with (2) what truly needed for the as-built structure to sustain the harsh operating environment

Challenges whether seen or unseen, new or old, conventional or unconventional could turn up from time to time from project to project; besides, certain varieties of that can blend in so well with the norms; that often caught many of us – with no exception, that includes numerous *self-appointed yet inappropriately appraised experts* as well – off guard

Sometimes making sense of the consistency-inconsistency in the treatment to so many age-old **CRG-Specific** issues can lead us to nowhere better than the result from a blind touching an elephant, or that much like looking through the blurry lens of a good old kaleidoscope. Our mental verdict over the seriousness of challenges we came across could at times be much more translucent than what actually appeared, or could just be the opposite.

Take it from what might come to pass during a typical **CRG Engineering Venture**,

> While on an assignment maintaining the pace along with the project progression, things can happen for whatever reason, only problem is:
>
> > Should any important issues were (inadvertently or not) *excused or overlooked and were not properly resolved* right then and there, more than likely those unsettled issues would push on and reemerge to become **brand new** again
>
> That for sure would catch up with "someone" when time comes; perhaps with stroke of luck if not in our own face during our own career lifetime, but, certainly would fall onto the shoulders of some latecomers someday

As many Structural Engineers would have agreed with what are exemplified hereinafter once assimilated through practical experiences:

> Some of the worst **CRG Challenges** tend to pop up more frequently while handling **Rehab or Replacement and/or Upgrade projects** (**RUP**) matters than otherwise
>
> By coincidence or not, what can make the situation worse?
>
> > Many "not so trivial" issues were off targeted as superficial in nature <u>at first pass</u>. The impending risk from not realizing the seriousness of the matter can lead to a major **RUP** setback when the situation was not properly dealt with early enough
> >
> > The weigh-up on a specific **RUP** issue being trivial or not is very much *site specific, task-dependent and multiple-conditions-based*; it could hinge on many factors such as which party is responsible, which discipline is riding along or crossing the critical path, and on which party is on the receiving end catching the hot potato, etc.
>
> Experience also suggests, rarely ever two **RUPs** are identical in each and every attribute;
>
> > Thus a much wiser engineering strategy should be, taking every project as unique challenge rather than brainstorming over how to cut corners through imprudent means, or for worse to skirt around scamming an <u>unchecked</u> copy-and-paste scheme for quick bucks
>
> As a result while accepting a freshly authorized **RUP**, <u>unless</u>:
>
> > We, at project management-engineering-contracting level, (1) had fully comprehended the implication of each and every identified issue and concerns associated with the project, and (2) had generous site-specific knowledge with confidence in how to overcome site-unique muddles inherent in the **RUP**, or (3) for the better of all if we had effectively conducted "similar businesses" in the past;

Otherwise:

> Our pseudo-expert like attitudes taken toward these *up-to-the-minute* tribulations that demand *much higher-geared technical* treatment could make us act out more so blatantly ignorant or feel more so confounding, which presumably could be attributed to (1) grossly misreading the problem nature or from (2) lacking expertise in handling **RUP-related** crises or from (3) lacking needed skills in resolving **CRG-based** structural issues in general

Take a developed mental aptitude usually adapted through direct hands-on project involvement:

The sensation engrossed from treating or mending ailing **CRGs** innate in many **RUPs** is rarely or never a pleasant one; no need to extrapolate further to agree with, the aftertaste from taking on **RUP**-flavored challenges should be much bitterer (and much more overpowering) than from designing **CRGs** as brand new entities for a fact

Oftentimes, once setting our feet deep into the **RUP** undertaking, there is no comfort zone within the project confine to take cover from a variety of challenges combining engineering, fabrication, erection safety and beyond. It could even be more unbearable when driven into unanticipated time-crunching situation, for instance, meeting a move-forward production/shipment deadline or a hurry-up of *furnace shutdown-startup schedule* and so on

Just be prepared; imagine on top of coping with as-assigned/as-accepted challenges, it can't be more woeful than (1) expending extra resource shelled out for precipitous on-site surprises in an actively running Mill or (2) sharing a third parties' misstep/mishap that grew to become an add-on to our own **RUP** project responsibility, etc.

<u>Many Desk-bound Structural Engineers may have very little or no clue as to why and how certain issues accompanied with so much negative vibe could transpire in so many ill-fated older Mills.</u>

The cluelessness could very well be stimulated and accumulated from (1) imposing sketchy or unsophisticated engineering knowhow to qualify/certify the adequacy of **CRG** structures during the initial design session; and more so afterward from (2) lacking **RUP-Critical** proficiency in the main as to "seeing into" and "seeing to" those **RUP-Unique CRG Challenges** *one on one, one by one, face to face and eye to eye,* that said, *let along taking full control of it all*

One of the mysteries in need of earnest rationalization is in how and why so many good old massive strong-looking **CRGs** could end up in such a raggedy shape as forestalled in so many **RUPs** of late

Actually, many engineers – self-motivated or those obliged to bringing forward good reasons why certain bad things happen – had never "seen or been told of" the tale in actual existence how much calamity some of those **CRGs** must *sustain* in active service

> To have a feel for how bad things had been, making a trip to an ill-fated Mill is the simplest way to prove to themselves especially those skeptics
>
> Once there, one can try an on-site self-test to make sense of, for example, *whether if the girder is too weak against torsion despite being overly strong against flexure* among other "shortcomings" such as lacking adequate strength against metal fatigue
>
> To have a much better picture of how critical the situation had been in certain Mills, just ruminate the level of real-life mishap there is after "being informed" that, there were *miles after miles* of crane runway girders in dire need of "earnest overhaul"

In one of those unimaginable but believable instances, as of this writing, the totality of distressed girders had exceeded 16 miles in length; yes, *16 miles long* all identified in a single facility

All in all, that is not a laughing matter to the Maintenance and Operation crews, their everyday routine could be like tiptoeing through tulips and wondering when the dreariness could end. *The scary part we do not want to know is knowing how many other facilities has been suffering from similar or more (or less) severe plight*

Accepting **RUP** reality is plentiful of an excitement, which might bequeath a unique challenge or agitation to so many desktop-based structural engineers, especially to those not fully prepared for this kind of action-packed adventure, which involves utilizing not only good engineering skill but also good inspection skill as well. Just envisage if taking on a **RUP** without a reliable diagnostic based on proper evaluation of accurate/accountable inspection result then, how can we expect to offer a cost-effective fix *fast enough, cheap enough and good enough?*

All that said is a good enough reason why **RUP** is not a routine undertaking for all comers. Just ruminate a fact, most – if not everyone – Desktop-based Engineers being assigned to carry out inspection duty to (1) make "observation" and (2) collect "evidence" had real hard time adjusting to the working environment in the first place, to a much greater extent, most of them are short of essential knowhow to "look for" issues or problems through a Millwright's perspective.

The unfamiliar hustle-bustle environment in the running mill could easily knock against a person's confidence almost every single time;

Once put up to a fresh inspection assignment while walking the aisle or after climbing up on top of a girder, to many engineers doubled as makeshift inspectors, newbies or seasoned alike, they could be so overwhelmed at the never-been-explored setting, and could very well start off with an uneasy gasp then act as if questioning themselves with something like "what the hell am I looking at?"

That is no surprise; although similar inkling can be echoed from so many among us having been through the same situation, but, it is not all that bad as showcased once getting the hang of it

It's not all that bad? Yes, provided we possessed a millwright-ranked visual-spatial ability; such a special status can be earned, but that must be vetted through unbounded must-have **RUP**-driven experience, only from fielding activities such as "treading" the sooty ground, "climbing boldly" up and down, "feeling" the heat, "hearing" the startling impact, "absorbing in" the uncomfortable noise, "smelling" the greasy air and "breathing in" varieties of particles, etc.

Field experience as touched upon earlier can never be vetted through sitting in front of a desk.

Even so if proven proficient and field qualified for the task is still insufficient, because besides that, the engineer-inspector must be prepared to shift gear on the fly with dissimilar mentality to face fresh round of **challenge** when walking into different facilities – in carrying out the as if never-experienced assignment – for examples, Plate Mills, Casting, Rolling Mills, BOFs or Shipping Bays, etc.

Finally back to the desktop viewing from a desktop engineering perspective, for certain on top of dealing with the typical one-of-a-kind **RUP** despairs, as a never-to-forget reminder, a worse fate – may not be the worst yet – may come from caring for stricken older **CRGs** having sophisticated **Unsymmetrical Section or Arbitrary Profile Geometry**

For better or worse, it sure beats any other form of professional rewards from being able to relish invaluable **RUP**-themed specialty experience – bitter and sweet – through and through and be successful while doing that. But by and large the more we had endured through overcoming these **Challenges**, the more so we may come to a solemn realization that,

> *Something must be seriously wrong all along owing to ill-treatment to **CRGs**; be it a reality or fantasy, but not many would openly acknowledge or speak loudly about the "unpleasant" issue outright, or simply ignore the grim facts all together, but;*
>
> > *Is there a problem with the maintenance upkeep?*
> > *Or is it a problem with olden not-so-trustworthy engineering treatment means?*
> > *<u>Or is it both</u>*?

<u>On questioning the questionable and pseudo-unquestionable reality:</u>

> In times gone by, and counting up the events through so many **RUPs** forward to the most recent episode, just picture as if we were able to see through the kaleidoscope with a gleaming twenty-first-century versioned lens and identify that -
>
> > *Some of the well-accepted well-matured so-as-fully-trusted good old ways of qualifying **CRG** (especially the **Unsymmetrical Sectioned** ones) so rampant in the mainstream were **flawed** in so many ways, which, surprisingly yet with no big surprise, are still able to take hold "universally" becoming a formidable deep-rooted engineering "tradition" – a bad tradition to **CRG***
> >
> > *By default, many unwary **CRG** engineering-design emulators were taken in impulsively to "embrace" and "put up with" the shoddy "bad tradition" and let that be so cherished, thinking things must have been working "perfectly-in-disguise" with what we have inherited so far so good; therefore, never should we question the "traditional" practice – that actually is not so far so good after all*
>
> But that deep-rooted tradition had already been proven no longer acceptable as to fully satisfying the contemporary and futuristic millennial's design requirement at large
>
> Why is that? Funny so sounded but not so funny a question to ask though; if the good old **CRG** engineering treatment was "flawless" then why so many deficiency findings were exposed out there?

<u>On balance:</u>

> There is no gain except inducing more harm to the Industry if we continued denying the pesky reality that had, time and again, casted out in front of us
>
> Many desk-bound Engineers who had never set foot into the Mills or those who had no idea of what a good Inspection Report looked like may ask again, why is it?
>
> > Simply because numerous **CRG-themed** distresses had long been branded on record in so many already-been-challenged facilities that needed our attention, not the kind of incompetent or spur-of-the-moment attention but serious-engineering-vetted attention
>
> *Once again with regard to the flawed design tradition, we should not be safeguarding the engineering-incorrectness and putting on engineering-flavored political-correctness at the same time or any time*

So for fair argument's sake from speaking out for the good of the Industry, If other than censuring the plain-to-see incorrectness or putting zero blame on what transpired hitherto such as flaws in engineering-and-detailing practice or slack in Mill's upkeep maintenance, etc. then again, who is to blame? Ask again, where did these **CRG-themed** distresses come from?

Quite obviously by now we all can "guess" correctly who is to blame the most, only if the guilty party concedes even from an outsider or a **non-engineering** viewpoint; so dare anyone to act too politely to "not see" that these **CRG-unique** structural distresses, **RUP** issues, site challenges, repeated-repair-to-no-avail, puzzles and faults were not rooted from **ill-engineering**, agree?

If the answer is a sincere yes then, we need a good explanation and a good resolution as well

But prior to developing a much improved system to conquer these challenges or solving the ongoing engineering conundrums, we must come to an agreement with the fact; there is a big problem out there. If still not so convinced, but for the least, we could confer a more prudent stance that may entail a couple of sensible questions along these lines:

- *Targeting on those deep-rooted longstanding design "traditions" that had gotten away with taking-it-technically- easy for too long – for instance, **avoid the dealing with shear center's influence** – and although that had worked out so conveniently for <u>structures of **non-CRG** ranks and classes</u>,*

  *Should all that be examined and reexamined prior to furthering our full-hearted endorsement just for being proactive for **CRG's** sake?*

- Or else*, shall we continue on copying and pasting those same old schemes step by step as is, and stay put at where we have been?*

  *Further to that, should any longstanding engineering-design procedural steps, code stipulations and design guides, etc. <u>be questioned</u>?*

  *Do we need additional clarification or fine-tuning on **CRG's** behalf, in a more straightforward manner?*

<u>A sensible reflection on what really matters:</u>

On one hand, we owe it to ourselves big time to uncover (not a few but the whole shebang of) reasons why some of the traditional engineering procedures worked fine elsewhere should become inadequate (or insufficient) for **CRG** application in certain aspects

> Why? Again, simply because *<u>structures of many other ranks and classes</u>* (notable one such as bridge girders) are not entirely equivalent to **CRGs**

On the other hand, if no one agrees with what had already been emphasized up until this moment and try one more round of kicking the can further ahead then, too bad and more than sorry to say, we would be trapped in the same old rut all over again, unless we audaciously upend the **CRG Engineering Procedures** from the bottom up

> Just imagine as if we were redoing an outdated engineering-flavored eatery, to be fruitful for which, we need to remodel the kitchenette, add new gadgets and fresh ingredients, modernize menus and recipes, and to come up with value-added ways in how to prepare and serve (our Clients) the Industry better, etc.

What went on and on so harshly so far has been targeted only at **CRG Engineering Procedures** in terms of classroom-typed pure technical sense, but the challenge we supposed to win over does not stop there. The flip side of the challenges beyond what we had publicized can come from the Mill's Operation that we also needed to conquer over.

Remember we mentioned **RUP** muddles, puzzles and faults, etc. a while back and wondered what can that be and how bad can it get?

> Imagine how we were challenged/challenging ourselves to fix the mess under the extreme **RUP** pressure? Take this one-of-a-kind **RUP** for thoughts:
>
> > What could be our immediate reaction during an initial site visit after "seeing" ten girders in a row, all of like geometric attributes to begin with, but went through with ten different foregone repair-design schemes, executed under ten different contracts/engineers by ten different contractors, unfolded over ten different occasions, overseen by ten different contract administrators?
> >
> > Believe it or not, but what illustrated is an example of no fluke real-life **RUP** challenge
>
> Imagine once more when running into some smaller-scaled surprises in disguise; how do we feel, and what should we do (1) after finding out the end restraint is missing a positive tie-back to the building column or (2) finding out all the bolts were missing at the top flange connected to the thrust plate? Do we just leave it alone or call for a safe shutdown?

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<u>Anyhow in view of what's emerging hereinafter and throughout the upcoming **Chapters** as well, some of the bits and pieces of the (interesting) info yet to be dealt with thus far may appear as some fresh revelation to those unprepared.</u>

> What pronounced out loud is not to impart any disrespectful hyperbolic message to so many self-motivated **CRG** enthusiasts, but to prime up their expectation over what is yet to be unwrapped. Nonetheless prior to embracing any of these soon-to-come but not-so-old-fashioned **CRG Engineering** ways and means for practice, it is far more important for us:
>
> - To resettle our technical awareness and get prepared to face the so-called "**seemingly new but not so new technical challenge**" and
>
> - *To come to realize that, what would be identified later on as if brand new for the occasion is really not that new at all, but actually age old "advanced engineering commonsense" being reinterpreted in ways to provide more details that were <u>missing</u> in the "Books" all along*

With aims of authenticating such ruling, we need to start/restart from square one – yes indeed, here we go again starting from square one – sounds familiar? Only then all those issues and challenges could be appraised with greater respect through tidbits of renewed understanding drawn from heaps of rehabilitated wisdom.

## 5.2 Crane Runway Girders Are Not Bridge Girders

Many Structural Engineers intuitively associated **Crane Runway Girders** (**CRG**) with **Steel Bridge Girders** on the basis of (1) the like appearance and (2) the comparable role in supporting "heavy" vertical **Y**-loads imparting on the respective structures.

In a way, the aforementioned awareness is not entirely (not) incorrect; but the scope and the criteria drawn on for the comparison is definitely not the fullest.

To avoid further misunderstanding and any unnecessary dispute, all it takes is expanding our cognizance range broader than those outward attributes. It should become much clearer to "know" that there are distinctive dissimilarities between the two entities with respect to:

- How they were framed,
- How they received their loads and,
- How they reacted to loads, etc.

*Herein to make obvious of our focus in this regard, the girders of interest are of open-sectioned; for that reason, boxed girder does not play a part in this discussion.*

(a) **Steel Bridge Girder** picks up its share of **Y**-loads mainly from vehicles trekking from above – streaming along either transversal or longitudinal axis as directed by the traffic lane orientation – in addition to supporting its own self-weight and various **X/Y/Z**-load vector resultants and/or components as applicable as (1) from the tributary share of loads passed on through associated framing/connection and (2) through ambient sources or other environmental means, etc.

While not at an advantage (or disadvantage) as those intentionally designed as being standalone (such as monorail-supporting) structures, the conventional bridge girders in more familiar applications are not standalone, again, unlike some of those supporting *monorail track*s.

In contrast to crane runway girders, typical bridge girders were well integrated in their local surroundings with other associative and supportive constituents – such as stringers, decks, lateral ties, cross-lacings and/or bracings – in ways by design that interspersed into a relative chunky **XZ** gridiron/planar element.

In essence from a global perspective, leveled **XZ** plane as such does function much more robust than using detached framing arrangement for a fact.

> As prescribed by the as-designed framing schematic, the load responses amassed from sub-framing that scattered throughout the **XZ** plane were consolidated following a rational load path that eventually leads to the outlying primary girders – usually having heavier set than most of interior framing members – which served as the principal pillar/support element for the entire deck/module prior to transferring the rightful amount of absorbed load-reaction onto the supporting piers/foundations

*Herein the outlaying primary support girders are assumed to be fringed along **Z** direction, although in general that could be **X**-fringed or **Z**-fringed, which is vastly depending on (1) the elected orientation of framing grid orientation or patterns and (2) the eventual bearing locations/arrangements.*

By (1) strategizing the main/primary bridge girders to span along the **Z**-fringes and (2) implementing with as many as needed **X**/**Z**-oriented secondary members in between the fringes, and through (3) engaging the wide spreading of decking surface median that spans/spreads across the **XZ** plane in between, what ended up there becomes a rather sturdy **XZ**-diaphragm from an in-plane lateral load resisting point of view.

On taking in the primary **Y**-loads plus any accompanying secondary effects that may come with, and through the multitude of sub-framing that makes up the load transferring system, the **XZ**-diaphragm can effectively:

- Accumulate applicable **Y**-loads from multitude of wheels/objects, which could be in motion or be immobile and that be scattered randomly, in patterns, in queues, in isolation or that as clustered over the diaphragm

- Distribute and redistribute orderly in progression among all structural components partaking within the diaphragm in accordance with the hierarchical order established on connected framing sequence

- Dispense the eventual **Y**-load-induced effect – bending and/or twisting – apportioned from **XZ**-diaphragm through transitional framing into the fringe girders

For a diaphragm intended mainly for supporting traffic lane loads, it calls for a flat/leveled surface free of intentional openings or voids. Therefore all **Y**-loads originated from vehicle traffics that follow the natural load path onto the bridge girders along **Z**-fringes would share a common feature:

*Pointing predominantly downward and only coming from **Y**-source <u>above the decking surface</u>.*

A profile cutting across the **XZ** deck/module − as the transversal slash is made perpendicular to the fringe girders − reveals a composite section outline geometry that may resemble a fork prong-like or a rake prong-like figure, depending on how many girders/prongs there being integrated into the diaphragm framing.

One of the notable characteristics of the **XZ** plane to appreciate is, when placed under intended load, all girders are working together and helping one another through diaphragm action.

> With framing arrangement as such, automatically, each and every girder can be considered as fully supported at the respective compression flange once it was firmly affixed beneath the diaphragm's bottom surface

In consequence, no girder can freely behave on its own along the **X/Z** plane without bringing the entire diaphragm into action.

*Because each individual girder had been integrated into the diaphragm, it becomes clear that (1) qualification of provision against lateral torsional buckling and (2) design inclusion of effects attributed to loads off its **shear center** is no longer relevant.*

(b) <u>**CRG**</u>, on the other hand, is exceedingly underprivileged.

In the first place, it has to be configured as a stand-alone entity; and that sometimes be bequeathed with a not-much-of-choice characteristic to defer to, all that is vastly constrained by the unique Mill Operational-functional requirement.

**CRG** is basically a **Z**-oriented stick-like member to start out with; then in some of the more "elaborate" schemes, a bare **CRG** might be furnished up into a **2/3D** truss/frame-like profile by adding supplementary branches, attachments or bracing elements, etc.

Whatever the concluding profile configuration as furnished or outfitted, a typical **CRG** − with or without supplemental add-ons − when viewed from a *service efficiency* standpoint, for it lacks a genuine **XZ**-diaphragm much as bridge girders do, it is still very much a one-dimensional **Z**-oriented linear entity.

To accommodate overhead crane bridge span optimized for normal operation, **CRG** is strategized to line up along the outermost **Z**-margin of the service bay space/volume/void, which the crane/trolley can securely operate within.

According to typical setup as in most Mills, the girder fringes together with any protrusion thereof must be absolutely clear from the trolley service bay volume/space in between, and be totally free from any interference to the operation. This effectively placed a fairly stringent threshold limitation to the flange width of runway girder in order to avoid interfering with any crane components.

To acclimatize the hoisting movement pointing along <u>upward-downward</u> sense, all **Y**-loads must be "served" from below the **XZ**-service plane. Therefore to all **CRGs**, no **Y-payloads** could directly approach or dangle from above the crane rail-crane wheel interface.

*One of the distinguished features to* ***CRG****, there is no such thing as diaphragm*

For being a simple **Z**-stick or even being outfitted with thrust plate and/or associated elements, **CRG** always brings on extra technical hardship to Structural Engineers in qualifying against loads coming from both **X** and **Y** directions with eccentricity, all for a simple reason:

*It is never out of the shadow of its* ***Shear Center***.

In summary,

Some of the bare steel bridge girders – when isolated from the diaphragm − may look quite similar to a bare **CRG**, but there are big differences in how the **Y**-load vectors were pointing from/into the **3D** service space, i.e. one from above versus one from below. Besides that, lateral **X**-loads could be absorbed much more effectively by (bridge) **XZ**-diaphragm than by a standalone (**CRG**) **Z**-stick.

Fair or no fair to the engineers, it takes much more to qualify a **CRG** not only because it is different from most *structures of other ranks and classes* in terms of load nature but also by the fact that **CRG** has to defend various load effects – bouncing, rocking, rolling, swaying, twisting, warping, etc. – involving *all six degrees of freedom* all taken by itself. Thus no matter what we think; **CRG** is way outside of the realm of reaping benefit from **simple bending**.

## 5.3 A Humble Restart Once More

To show our utter sincerity off to a humble restart, we should (1) put our personal opinions aside momentarily and (2) hold off critiquing on several matters of concern such as complaining how **CRGs** were treated inferiorly in the past or arguing how should **CRG** be better treated from now on; but even so, isn't that enough a show of being humble?

Moreover, had we not been invited to visit any Mills or had never participated in any **Rehab or Replacement and/or Upgrade project** (**RUP**) undertakings then,

Do we realize there is a long-standing problem all along?
If so then can we identify the problem?
If so then what should we do about it?

Any way we slice it with; the commonsense assessment should narrow down to all three yeses, wouldn't it? Nonetheless, many indifferent naysayers would find ways to deny it all, problems or no problems.

With that frame of mind imparting in our day-to-day business as usual, perhaps, there may never be an *indisputable magic-wand-like engineering approach* that can facilitate the best resolution to each and every **CRG** issue out there – especially to some of the extremely challenging **RUP** issues

With luck lest it were too good to be true, somehow somewhere there is such *magic-wand-like engineering solution system* exist, then should we blindly fall for the not yet vetted pitch without a test drive and double-checking?

- Is the system truly *all-purposed* ***CRG****-adequate* or some *half-cooked-half-raw* phony?
- Does it assimilate to the diverse engineering mainstream's **CRG** treatment sophistication?
- Would the solution outcome align with or against the Mill **RUP** Management expectation?

Anyhow, taking a position (1) whether confronting the **RUP** challenging issue head on for a down-to-earth resolution (2) or evading the must-solved problems with self-approved self-certified engineering excuses (3) or some other out of the ordinary approaches such as *doing nothing* is a personal choice.

Other than the do-nothing approach and in fact, whether a rightful choice made or not depends largely on (1) how our technical pitch (or alibi) makes better logical sense (or nonsense) in the long run and on (2) whether the choice works out better (or worse) for all occasions

One of the Catch-22s unique to **CRG** engineering-design business-as-usual is, it is never a business as usual for good reason. Typical to any given **CRG** under our care, no matter what amount of resource/effort were handed over upfront; yet no one can foresee the **CRG's** future path whether toward a destined good or bad ending, why?

Because it takes considerable amount of "time" lapsed in order to confirm the long term performance score prior to showcasing any engineering proclaimed adequacy outright

And most of all, it is impossible to predict – hopefully not the case – the exact timing when a freshly minted **CRG** were to be transmuted from being a profit-making instrument into a profit-breaking burden is an impossible feat

As we had been inundated with so much **CRG-branded** dreadfulness so far so doomed, it seemed there is little prospect to pull off a total victory any time soon; but to speed things up on a positive note, perhaps from taking advantage of the technical advancement accumulated over the past few years and decades, it should be possible that an honorable mention can be awarded for trying, would it?

The continuous enhancement in facilitating useful (sometimes not necessarily100% helpful) **Engineering Tools** – *in terms of hardware, software, design codes and design guides, etc.* – catering to solving structural engineering problems as a whole is a well-established fact, especially those packages aimed for boosting productivity alone

But in spite of which, the benefit of utilizing some of these tools (or for lack of proper tools) in connection with improving **CRG's Long-term In-service Performance Quality** seemed stagnate if not getting worse for several reasons, and so we shall see

On matter of receiving criticism, the tendency by natural ways of shared human reflection is to counter react whether or not the target of blame had exhibited any poor engineering-design judgment or instigated poor-quality engineering results, etc.

The truth is, many among us failed to recognize poor-quality engineering results can be caused by application of "wrong" conjectures in the beginning – such as *pinning the shear center at a wrong location or ignoring the effect of torsion* all together – and/or *from misuse of tool or using the "wrong" tool to begin with*

That said however, it is never a good situation to be in from trusting impoverished ways and means and be oblivious to the adverse consequence; or spreading *misinformation* and maintaining a blind eye to the harms it caused. The worst is advising or replicating the *misinformation* to the mainstream practitioners for them to follow and letting it take hold incessantly, especially when:

- *<u>Not realizing</u> that a certain methodology rated as "good or good enough" long ago may turn out to be "not good enough" or even in some way "seriously wrong" some years later, and*

- *<u>Not knowing</u> the relevance of certain **Engineering Tools** or **Engineering Presumptions or Longstanding Assumptions** were misunderstood and/or misapplied all along, etc.*

Anyhow, many enthusiasts with all sincerity and honesty to forge ahead and those destined to be experts/non-experts must have "suspected" one thing by now,

There ought to be **CRG**-Engineering related predicament, shortcomings and all that not being publicized or casted openly in black and white just yet – hint: **torsion and fatigue**

Knowing there is problem and solving it are two different events; it might come down to how much knowledge we had on how to have the problem solved. Anyhow, all the **CRG** enthusiasts should get the drift, that some of those not so notorious challenges could be buried along the way throughout any stage(s) of project development while carrying on with variety of **CRG** assignments upon:

- *Designing* these (unsymmetrical-sectioned) structures as new,
- *Investigating* the existing ones for cause of failures,
- *Mending* their repairs to address known (or unknown) structural deficiencies or
- *Upgrading* them for better performance or greater lifted capacity, etc.

As always, **CRG** being identified as a potential prey subjected to torment from **torsion and fatigue** is not an overstatement. The sophistication veiled behind the diversity of assignments like those is never a "good-old" or "same-old" in nature all because of **torsion and fatigue**. To get a somewhat modest perspective in abstract sense, a "same-old" issue may take only five pages of calculation to resolve while **torsion and fatigue** may take one hundred pages.

During typical engineering process involving **torsion and fatigue**, the proficiency in handling of data emerging from the as-required intermediate sub-processes is a good measure of how capable the data-handling-collection system is

In terms of what needed to qualify the design of **CRGs** when subjected to flexure and torsion, easily we need to deal with **seven** categories of structural responses and at least **eight** different brands of stress. Therefore it would be a big mistake if mistaken which as no-big-deal "simple bending" varieties and that not being treated readily by means of robust data management utilities

In other words, we realize how cumbersome that is only when doing all the must-do chores catering to **torsion and fatigue** *in a proper way*, *and then just do it to find out*,

At some point while executing the problem-solving analytical procedure, once trapped into a much deeper heart of the matter, we see the heaving of volumetric information launched after processing those "good-olds all-engineering" matters can vastly accumulate the transitory "junks" so massively that can so quickly overburden the (mediocre) data handling facilities, and in no time producing atypical non-engineering natured strength of "newer" challenges

Many engineers do not realize in order to deal with which it calls for advanced technical ploys coupled with properly elected "non-engineering-cognizance maneuvers" to overcome numerical data related obstacles sourced *outside of engineering confine* even though the goal is solving pure engineering-purported problem, *and then just do it to find out*

So what to do with obstacles sourced *beyond engineering confine*?

**CRG Engineering Business-as-usual** should be looked upon as business-not-so-usual to begin with if doing **CRG** business the right way. To cover all bases without missing the cue in the engineering process, we should be prepared to handle the coming of tremendous amount of waiting-to-be digested numerical information throughout every stage *all the way to qualify the design of structure against material fatigue*

Wondering humbly, where does that come from?

Have we not been called to mind among all things we have to mess with, there is a special phenomenon called *moving load*?

Keep in mind, the conclusion of a relatively short-termed grueling path through "moving load analysis" is only the beginning, the breaking point is anticipating when the "data swamp" would gain the upper hand to complicate things further, and for a much longer haul, too

For many reasons as said, one might need to carry on with dissimilar tactics to solve **CRG** problems. Surely it calls for some thinking and planning outside the box, but that is all normal and should not be dissuasive enough from not trying, simply because the overall **CRG** problem solving strategy is not much different in principle with respect to making appropriate judgment and choosing (1) the right *problem-solving **method*** and (2) the right *Engineering-analytical **tool***

Electing right tool(s) is "extremely" important − for the benefit of entire project − especially for dealing at the most fundamental level for those ostensibly routine *must-do chores* that must be completed as part of each and every pre-analysis and post-analysis setup process

Granted that during normal **handling** of those universal *must-do chores* – such as load definition or geometric model making − even though on *basic engineering principles* the exercise may appear deceivingly "simple and routine," and although the same **simple-bending-based process** had worked out justifiably **OK** most of times for numerous **non-CRG** applications, and yet, it may not necessarily fit for all comers and so we know; don't bet the full basketful of simple bending frame of mind on **CRG** matters. *The so-called **CRG** matters go way beyond the literal meaning of pounds or kips, feet and inches, etc.*

While focusing on the generic dealings with **CRG**, **symmetrical or unsymmetrical sectioned alike**, there are two special areas of concern that must be attended to from a pure *structural analysis viewpoint*:

- *Distinguishing* what comes before the dealing, and
- *Understanding* what should come after the dealing

First on the "**what comes after**" part − the most important part:

It is about the level of mutual **adaptability** as that harmonized between the analytical tool(s) and the solution methodology; out of whichever parallel pairing between the tools and methodology, once the combination was committed and adopted, we must offer a satisfactory answer to this ever living universal question:

*Would the combination pair –* tools and methodology *– turn out reliable results agreeable with the genuine structural behaviors that involve countless complex crossing points between **flexure** and **torsion**?*

Then for the "**what comes before**" part,

Which is a matter of pre-analysis information handling strategy setup that many of us (even among the most conscientious ones) might find it surprising after evaluating our engineered achievements on our own scale as measured between (1) how far off away from the model-building objective and (2) how right on target had we been as to some of the very familiar themes on **CRG** terms, for examples:

- The subject of **structural idealization** in much greater breadth beyond these scopes:
  - Material properties
  - Dimensional measures of structural components
  - *Boundary conditions*
  - Cross sectional shapes beyond **I**-section, etc.

- The **loading effect** augmented beyond the definition of simple crane wheel arrangement and loading magnitudes, etc. – nominal **P** and the companion **δ**

<u>To get going with a humble restart, a number of forthcoming sections/paragraphs in the Chapter would touch on perhaps the most **misunderstood topic** of **CRG** interest that were rarely initiated elsewhere as of this writing; along the way with confidence that should lead to a better **understanding** of what it takes to idealize a structure for (1) analysis without missing any vital loading responses including **P** and **δ** and (2) the proper handling of associated effects – local buckling, torsion, stress combination and so forth.</u>

## 5.4 Looking Into the Environment Where CRG Is Living In

Imagine the engineering life being reverted by a century; back then the tools for the trade were kind of primitive, certainly not as aplenty in quantity and as versatile in quality, compared to what have we being offered by many familiar tools available in this day and age.

Moving forward to present time,

> We see quite a few <u>self-adapted</u> "specialists" firmly believe, through some of today's do-over reasonable/available means incorporated in selective tools, there shouldn't be any problem in this world that cannot be solved
>
> Yet, even without considerable attainment proven to themselves personally, some would "actively promote" the <u>self-adapted</u> practice as if they can flawlessly (1) analyze and qualify the design of any given structure – such as **CRG** – and can quickly (2) bring forward a great deal of equitable results to back it up with relative ease, then can rightfully (3) declare an overall success with plenty of <u>self-assurance</u>

What just professed could be much "too harsh" or "too mild" depending on how we look at it, which is debatable, yet whether the claim is an overstatement or not is what we have to find out in a while; the resulting pros or cons is not meant to be judged for all purposes and intents, but, it is opined solely on **CRG's** behalf.

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From here on out, without actually doing the real thing, the "wild" rationale on the subject is based entirely on our engineering commonsense.

> The main purpose is to "see" whether or not the analytical results and problem solutions brought on by way of specific class of tools does bring to light a <u>numerical harmony</u> that should be on a par with the true **CRG's** *structural behaviors* under intended loads

The *Structural Behaviors* being purported herein are by all accounts the true *Behaviors* as exhibited in typical **CRG's** real life.

> The focus is not just the spontaneous scenes caught in a spur of moment, but collectively those as chronicled through every eventful checkpoint observed during the structure's in-service lifespan – ranging from beginning (making practical sense by **what comes before**) to finishing (being realistic by **what comes after**) – and that is the big idea here

<u>Starting the **CRG** engineering-design drill on the right track:</u> Elect an "officially recognized" well-liked <u>tool</u> to get under way;

> Assuming all is so far so good and so well with a user-established **I/O** packet given to us, which were duly established (by someone else) in accordance with the <u>tool's</u> directives; initiating from there is presumably a faultless situation to start the conversation
>
> Having the as furnished **I/O** packet in hand, (1) regardless if with it the subsequent **CRG Structural Engineering Process** were creditably carried out or not, and (2) irrespective to what may come before and what comes after our <u>engineering treatments</u> were implemented, but, in the

end, achieving a total numerical harmony – only in numerical sense – has a lot to do with the elected tool with respect to how had it been prepared/planned ahead to handle these two good old effects:

**Flexure** and **Torsion**

While living in a **CRG** world, or that much as many **CRGs** *surviving* in their world, what concerns the most − not us engineers but the **CRG** structures in focus − is during our engineering process, what happens if "somehow" not knowing we "failed" miserably for whatever reasons?

Think not happening? Think again, for instance:

> Quite often we triggered **CRG** engineering/design fallacy (**even inadvertently**) as soon as we (1) *initiated unrealistic assumptions only to simplify an unforgiving complication* or (2) *took up unjustified shortcuts to terminate a lengthy/intricate structural qualification journey*, and still called it all good, irresponsibly in a way

As widely understood in the classrooms, it is much easier to segregate the topic of **Flexure** from **Torsion** on engineering mechanics principles for learning purposes than the actual messing with their *convoluted interactivities* emerged in the real life (and the design) of **CRG** members.

The fact applied to open-sectioned members is, whenever playing with the combined effects of **Flexure** and **Torsion** – there are generic shortfalls subsisted in the legacy engineering approach as we "knew" from *much as described in previous **Chapters*** − we must avoid furthering the latent misunderstanding of what were misunderstood to such an extent so deep-rooted in former practices.

At this point of discussion, whatever happened had already happened – as we can't turn back what happened, nor can we assume it didn't happen – there won't be anything productive if we keep on denouncing what's over and done without moving forward and taking appropriate action.

> Not to be judgmental while moving forward from here and on, it is important to give an honest answer to these questions:
>
> - Do we clearly "see" beyond our desktop what these two Engineering Mechanics Creatures (**Flexure** and **Torsion**) are (capable of) doing to each other on paper/screen and that to the structure in real life?
> - Are we "capable" of dealing with the "messy" structural responses owing to intended loads handily? And if so, can it be done with utter confidence to handle the "mess" without employing **structured data organization means** or without relying on **database management tools**?

Just envisage, in an audit session into the numerical jumble following a routine moving-load analytical phase involving both **Flexure** and **Torsion**, how "not" to go astray during an in-depth review of which is actually hinging on how well versed we are in a few simple nuts-and-bolts matters – reflecting how **CRG's** true feeling during mill production service – such as:

> - Identifying "correctly" the respective load response types – in global and local senses – there existed and the associated stress components induced by each individual phenomenon; *how many*? For instance, how many types of shear stress can simultaneously "attack" the cross section profile? In terms of numerical value, how many of which are reversible?
> - Understanding what are the possible harmful effects (excessive deformation in **3D**, material fatigue failure in addition to considering global/local buckling and material yielding, etc.) that might bring on to **CRG**

- Distinguishing how does each raw stress component came through (through proper calculation) and knowing where to stock up/look up/match up information (before and after proper load event permutation and/or combination); *how*?

Being able to characterize those "how" and "how many" concisely is a good start, yet still far from reaching the finish line for a big old reason,

> To emerge from our virtuous enthusiasm based purely on fancy engineering theory into experiencing and mastering heavy laden **CRG**- characterized number-crunching reality is a no fluke transition. And that takes some very specific disciplining to accomplish specific goal; we meant the data management kind
>
> On the face of it, as if an artificial intimidation is hitting hard to the faint-hearted ones or those unprepared. And yet, taking up **CRG**- characterized challenge is a very unique experience
>
> Without a properly developed tool to save the day, then it can befit treading over tricky numerical-flavored obstacles, struggling uphill toward a mile high technical highpoint; that be achievable only if we already have in hand the custom-tailored robust numerical data management facilities/utilities

Whether ready or not, there are other analytical-tool-related matters that must be considered and dealt with during the actual calculation progression.

> Once we were sold on adopting an analytical solution tool/package, with which to succeed or not would depend on (1) the usefulness of amenities that the tool had made available at our disposal and (2) our level of (mis-) understanding of **CRG's** true-to-life structural behavior. So as anticipated there would be success under normal circumstances, when all's well that should end well, too; hopefully so, but, there is a "but"
>
> *<u>Before</u> implementing a comprehensive* ***CRG's Defense Strategy*** *against* (1) combined wrecking from **Flexure** and **Torsion** plus (2) *all other probable modes of failure to be taken care of in one swoop through the tool duly developed/selected for such task, the heart of the matter to contemplate is (1) how to deal with the swamp of information churned out in wake of multi-wheeled moving load analysis and further from that (2) how not to miss any important too-numerous-to-be-counted in-between steps – such as proof of meeting serviceability requirement and provision of ample strength against metal fatigue and so forth – prior to ending the process*
>
> > *Then think about it more seriously, are we asking for too much from the tool or setting the bar too high?*
>
> *<u>Before</u> it's all said and done with the load/stress combination/permutation process, how can we convince ourselves (or the world) that the load conditions/definitions we had fostered are "realistic" – in relation with respect to both elastic centroid and shear center – and the result we acquired through the process had covered all scenarios in terms of how our* ***CRGs*** *feel in their real world?*

Prior to finalizing a painstaking data management plan, we should observe the fact that **Flexure** and **Torsion** were the standard not-so-enjoyable pastime germane to all **CRGs**. It pays to mentally reboot the know-how of these events at the grassroots level once again, we should have acclimated:

- The "effects" owing to flexure alone could be narrowed down to the "pure flexural bending" type, which often was resolved into an orthogonal set of **principal** global/local **X**- and **Y**-axes whichever is meaningful and <u>convenient</u> for better perception of "effects" such as bending moment $\mathbf{M_x}$ and $\mathbf{M_y}$ and also **flexure** shear force components along **X** and **Y –** (Question: What if torsion is also involved?)

All that should be easily understood, but when the focus is on unsymmetrical sectioned **CRG**, the orthogonal set may need to be transformed to **X'/Y'** to make better sense – remember **unsymmetrical bending**? Yet contingent on the type of stress associated with unusual structural configuration, we may have to deal with *post-buckled effective section properties*. After all, the sectional plane under flexure would always remain a flat plane

- Torsion deals with not only the "immediate effects" but also their "side effects" as derivatives from the rotation about the member's global **Z**-axis – which is the main trigger of non-linear **P-delta** response yet *to be covered in upcoming chapter*. It is immaterial whether the generic **Z**-axis is passing through the **Elastic Centroid** or the **Shear Center** (**SC**) at this point, but other than that, we could simply state that the rotation about **Z** is induced by a generic torque moment $\mathbf{M_T}$ which can progressively increase the ongoing **P-delta** until the value converges

In any event, however the **CRG** feels in its life in service is all natural, but per basic Engineering Mechanics, the structural responses to the applied load and/or their solution functions, whether on flexure or torsion accounts, be they in terms of **X**, **Y** or **Z** occurrences, linear or rotational, etc. were highly dependent on these entities:

- *Geometric Configurations* − geometric section properties
- *Material Properties*
- *Applied Loads*
- *Structural Boundary Conditions*, etc.

Each participating entity as listed does subsist in its own special place(s) in the analysis of structure of all ranks and classes. And for the success in any intermediate step of a trustworthy analytical progression, there is simply no room for the designation of any one of those items to be inaccurate or unrealistic, either physically or logically.

Yet from all considerations we can think of, "*Structural Boundary Conditions*" being the very last item as listed should deserve to be the most commanding entity among all, which in no doubt should have the most dominant impact on the analytical results of **CRG** applications, and we shall see why in a moment.

## 5.5 Structural Boundary Conditions – As Understood

Recalling the exercises of various Engineering Mechanics Problems:

While solving a *solvable differential equation* in the most classic/organic workouts (by the Book,) we typically arrive at the final (close-formed) results through **two stages**:

The *first* stage of process "deals" **primarily** with only *a subset* of the generic structural conditions (or preset constraints) while the *second* stage "deals" inclusively with (1) the *remaining* generic structural conditions plus (2) a number of specific structural conditions

Herein the action verb "deals" in *engineering sense* could be interpreted as "satisfies" with respect to the given condition in *mathematical sense*

As expected as in many textbook examples, the respective solution expressions (usually of multi-termed) obtained from each of the two stages are not identical when matching term by term, although the dispositions in the overall appearance in both are very similar

By structural engineering recognition, the generic structural condition consists of *Geometric Configuration*, *Material Properties* and *Applied Loads* while the specific condition has everything to do with the *Structural Boundary Conditions*

Results obtained from satisfying the generic structural conditions made up to only the first half of solution process; the reason being, some of the algebraic terms still carry *undetermined* constant(s) pending further designation, thereby it is regarded as unfinished or as semi-finished solution; in a way it is identified as "**generalized**" result

Whereas in the second half, what labeled as incomplete following the first half must be resolved by "satisfying" all the as given specific structural boundary conditions in order to arrive at a definitive (completed) solution

The process as noted above works out for all *solvable* boundary-value problems in wide-ranging practice; however, the so-called **generalized** solution in its raw form is, understandably, not ready for specific application intent and purpose

In other words, it is still independent of the boundary conditions. Therefore the solution to any specific problem (*solvable equation*) is never completed or finalized if not yet satisfying all the given boundary conditions

Consider solving *simple* structural problems during the course of Elementary studying of Strength of Materials:

Being *simple* much as in the dealing with most of single-spanned stick-like beams, for which very often we got by – for statically determinate structures only − with formulating and solving *simple* algebraic equation(s) through (1) free-body diagram depicting the *structural geometry, applied loads and the unknown support reactions* associated with a (2) suitable set of **XYZ** static equilibrium scheme, and then conveniently led to the final solution, usually in one single numerical take without strenuous effort

The false impression from obtaining an "instantaneous" solution at such ease as if already so compensated to the boundary conditions all along often misled or downgraded the importance of:

- Understanding what and how "seriously" can boundary condition truly affect the solution or analytical results in general

- Practicing proper steps as needed in setting up **problem-solution definition** specifically for **CRG** applications, etc.

Before the real Structural Engineering Design Task could start, whether the structural object/entity is to be analyzed based on simple equilibrium principle or from solving complex mathematical equations, we must have the **problem-solution definition** squared away − prior to even thinking of unraveling the unknown physical deformation states and/or the unknown load responses of that structure under intended loads.

Making the point across more straightforwardly:

Before jumping into doing structural analysis in a hasty, one should place focus in these areas as follows as applicable and then make proper preparation accordingly

- Level of details anticipated in our solution in terms of number of degrees of freedom
- Method of solution, closed-form analytical or numerical
- Level of details in the structural idealization and discretization, stick-like or **2/3D**
- Loading definitions, **Elastic Centroid** based and/or **Shear Center** based
- Correctness in setting up the proper formulation of governing equation(s) and
- Appropriateness in prescribing the **boundary conditions** or the **boundary values**, etc.

## 5.6 Structural Boundary Conditions – As Understood Once More

**Boundary Conditions** (**BC**) are prescribed at selected X/Y/Z nodes as *constraints* in connection with the specific structural entity of interest.

> Any **viable** solution **applicable** to the governing equation(s) must demonstrate that "all" the prescribed conditions associated with the nodal *constraints* specified at specific X/Y/Z coordinates had been met in one-to-one correspondence with the specific degree-of-freedom at these nodes; otherwise the resulting solution would be **incomplete** or for the worst be (**partially or totally**) **invalid**
>
> This universal position applies to all forms of analytical-engineering result – obtained either by means of manual calculation or automation – whether using direct closed-form solution schemes or that via numerical methods, etc.

But, no matter what we do to earn a passing grade, our analytical results can't be half right and half wrong at the same time. The bottom line is:

> *To any given real life structural problem, if the Boundary Conditions for which were set up unrealistically or wrongly then the entire solution would also be unrealistic or wrong*

In normal structural engineering applications,

> **BC** could be affixed to the structure at "anywhere" for which a certain aspect of the "physical states" or the "conditions" were already known − typically **BC** were applied (1) at the prescribed structural support(s) and/or (2) at special fringe that may interface with other foreign object(s) or external structure(s) that share a compatible set of **BC**

The generalized "condition" of **BC** could be specified through either the "displacement domain" or the "force domain." Yet most importantly:

> *The "specification" of these two domains of restraint condition should always be kept mutually exclusive; i.e. for a particular* ***degree of freedom (DOF)*** *in focus, if the constraining condition for which is already set to limit the "displacement field" then the quantity of the "force field" associated with the matching* ***DOF*** *should be left alone and not be redundantly set in any way*

> In other words, when "displacement/deformation" is already specified at a specific **DOF**, upon which the unknown "force/moment/torque or reaction" complementing the "displacement-deformation" must take receipt of whatever the consequence as unraveled at completion of the ensuing analysis
>
> > In this instance, the "force/moment/torque or reaction" is the natural after-effect matching what called for from the application of "displacement/deformation" restraint; and vice versa if situation is reversed
>
> Or simply put it:
>
> If the **displacement** – linear or rotational – at a node/joint along a specific **DOF** is a known quantity (zero or non-zero) then the **force** (or the reaction) along the matching **DOF** at that node must be treated as an unknown quantity to be solved for

While assembling the "global equilibrium equation set" for **Finite Element Analysis**, usually in the formation of each equation, for clarity, there is an equal sign tucked in between (1) the unknown parameters to be solved for and (2) those known parameters either given directly or that derived indirectly:

> Notice in the mathematical formulas established for solution through stiffness method, more often than not the "displacement" domain representing the entire collection of nodal **DOF** vectors is placed on the "unknown" side whereas the "applied force" domain is placed on the "given" side

In common **Finite Element Analysis'** recognitions:

> *The displacement-related **BC** is understood as "primary **BC** or geometric **BC**" while force-related **BC** is recognized as "secondary **BC** or force **BC**"*

Even for the same structural object when evaluated from dissimilar perspective as chosen, the associated **BC** may have to be specified and/or interpreted differently as needed depending on:

- The nature of the load application and the <u>anticipated</u> structural behavior and

- How the structural entity was to be modeled; ask, is that to be <u>idealized</u> in full or in combination of parts from using linear or curvilinear stick, surface, mesh, solid or hybrid assemblage in universal **3D** settings, etc.?

If a **Crane Runway Girder** were idealized as a simple linear stick then its **BC** would be:

- Specified through the "displacement field" and
- "Normalized" into two independent suites:
    - Flexural **BC**
    - Torsional **BC**

For linear stick members, catching up on the fundamental concept of flexural **BC** within its own suite (say, *hinged, fixed, guided or elastic spring supported*, etc.) should be a simpler matter, comparatively, and should have been clearly understood to all Structural Engineers thus no further explanation for that is needed here.

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And similar to the "simply supported" and "fixed" **BC** serving flexural intent, <u>most</u> linear stick members of torsion importance could be idealized into **torsionally** "simply supported" and/or **torsionally** "fixed" conditions as well; but apart from that, each designated condition could be classified further through its own distinctive savor for technical and practical reasons.

As understood as applicable to a stick-like member:

- Everything and everything else that has anything to do with torsion must one way or the other be established from the **XY** profile's **Shear Center**

- <u>*Unlike flexure, there is no such thing as pinned boundary condition in the world of torsion that deserves further explanation*</u>

*Many Structural Engineers had no idea that "Twisting and Warping" are **two completely different phenomena***

> *Depending on the Boundary Conditions, each incident could be turned on or off or that both be induced concurrently from the same set of torque moment whether due to a single load source or of multiple load sources*

*And therefore whenever setting up the torsional **BC**, these two measures could be "mathematically" restrained or released either independently or simultaneously whichever condition deemed appropriate*

Take an **I**-shaped **CRG**, basically the support condition for torsion would depend on whether if the **girder flange** (*both top and bottom*) at member ends is free to (1) move along which orientation and (2) rotate about which axis, or (3) otherwise able to twist and/or warp freely or not.

**Logically** besides being "free", these are several forms of torsion support condition per **Roark's Formulas for Stress and Strain**:

(a) Simple Support Case 1:

The end is free to warp *along* **Z** but not twist *about* **Z**.

It could be attained as close as to the real thing by providing (1) a number of seat bolts (not pitched/gaged into too many rows) at the bottom flange and (2) a one-directional or universal tieback link element at the top flange forming the mechanism provided that it could effectively *prevent twisting but allow warping at the girder end*.

(b) Simple Support Case 2:

The end is free to twist but not warp, which is the exact opposite to restraint intended for Simple Support Case 1. It is not practical or cannot be practically made to simulate the specific restraining intent and thus not recommended even if the condition could be attained from detailing standpoint.

Only if feasible then it may be more advantageous for certain special equipment components or mechanisms functioning under this mode, but **physically** there is no **CRG** connection detail that could intentionally allow the support end to twist and not warp.

(c) Fixed:

Obviously the girder end is not allowed to either twist or warp. It might be realized at the ends of continuous spanned members but not necessarily true. However, adding several closely spaced bearing stiffeners of relatively greater thickness at the ends of a simple span may be expedient in providing (to a certain degree of) restraint against both twisting and warping.

*In **CRG** engineering, the term "torsion simply support" should not be called "pinned" but if so insisted then normally it refers to "Simple Support Case 1."*

## 5.7 Structural Boundary Conditions – As Idealized

One of the most effective means to solve (solvable) engineering problems these days is through automation; despite the fact sometimes the result could be "wrong" from misusage, but it's still worth the while just to take advantage of better turnaround.

In any case, to minimize the chance of garnering "wrong" results, it is imperative to balance out (1) what were facilitated by the elected tools with (2) the quality of results we were questing for. This rationale applies to all classes of structures especially to **CRGs**.

> As far as **CRG** applications are concerned, to a great extent – as of this writing – there is hardly a perfect tool that can claim to be able to deliver a good-enough be-all and end-all solution from an all-inclusive engineering problem-solving and comprehensive design qualification viewpoint
>
> With an expressive understanding of unsymmetrical sectioned structure's unique characteristics, we should be very careful in how to match up our modeling strategy with the available amenities as *permitted* or as *limited* by the chosen software/hardware/tool

Tools whether acquired from vendors or <u>those developed ad-hoc in house</u>, just be leery that in some way, again – as far as **CRG** applications are concerned – all **finite element** based software tools have "modeling limitations" on the application of certain structural elements or constraint conditions.

> That may be a "defect" in eyes of many dedicated users, so let it be known that nothing is perfect except that, we should take note and be careful reading that some of those constraints were explicitly documented and some not

When not paying attention, those not explicitly documented constraints maybe where most of the "misusage" or "misunderstanding" comes from.

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*One can always choose whichever tool suiting the intended purpose but beware of what does apply and what doesn't.* For instance while preparing a model <u>involving torsion</u>,

> It would be "too bold" to **rationalize** the "modeling logic" with respect to specifying an *all-purposed simple support condition* (or simple restraints) with intent to "envelop" both flexural and torsional behaviors together without first qualifying if the would-be stipulated **Boundary Conditions** (**BC**) can be appropriately set for the application
>
> By appropriate intent we meant, in order to qualify an established collection of **BCs** to be all-purposed and to work properly under intended loads, the proviso must proffer a good-enough balance for:
>
> - The obtained structural analytical results from applying the **BCs** must complement as closely as with the actual structural behavior manifested <u>in the real world</u>
> - Regarless how the structural end conditions were idealized into the mathematical model, it must be compatible with the software promoted stipulation, with trust that the results would be consistent with what manifested <u>in the real world</u> through elected method of analysis, and
> - The as-detailed/designed **BCs** that gave rise to the analytical results should match up with the as-fabricated connection detail's real-life performance or the observed behavior in ways much as anticipated, etc.

<u>In an oversimplification as provided as follows – only to demonstrate what could be a serious problem in **CRG** application in general – the focus is on **BC** being specified through the displacement field (instead of force field) only:</u>

> A better starting point on the said issue could be led in from the making of structural models engaging only <u>linear stick members</u> (such as beams and/or columns, etc.)
>
> For being <u>general-purposed</u>, the software tool in use should facilitate **restraining** or **releasing** certain specific degrees-of-freedom at the member/element end joint to mimic various (boundary) conditions: Such as being *free, guided, hinged or fixed, or semi-elastic* through linear and/or rotational spring(s), etc.
>
> However it was done for actual **CRG** applications, one should be careful to discern between what is/are applicable and whatnot;
>
> > *All as discussed along these lines were appropriate for formulating equations based on flexural behavior **unless** the coexisting torsional effects were **explicitly** handled through software input and output **(I/O)*** means
>
> Consider the two <u>extreme opposite specifications</u> of **flexural BC**:

- On one extreme, restraining the **Z**-rotation **Degree of Freedom** (**DOF**) along with additional constraint of (1) **X/Y/Z**-linear displacements and (2) **X/Y**-rotations – or in other words, all six degrees of freedom per se – would constitute a "totally fixed" condition, which evidently applies to any applicable joint/node of interest provided that does not reposition under intended load

- And the other extreme is by releasing all six **DOF** into a "totally freed" joint

Whether it be totally "fixed" or totally "freed" per **XYZ-DOF's** all-or-nothing feature as specified, the **numerical implication in setting up the equations for Finite Element Analysis** from the said illustrations should apply equally to both **flexure and torsion**

That is so obvious, no argument there, except for the catch when "describing" the <u>Simple Support Case 1 in torsion sense</u> – *allowing warping but not twisting* − for which when considering the **Z**-related boundary conditions. Apparently we can't use the all-or-nothing specification for unsymmetrical sectioned members, would it become somewhat <u>in between</u> or <u>nothing in between</u>?

> So why is that?
> Isn't warping the culprit?
> Where is the shear center?
>
> How to allow certain support nodes – not all − to warp freely but not twist and satisfy flexure restraining requirement the same time is an interesting paradox. Is it possible for someone to draw a free body diagram showing what works and what not involving specification of all six degrees of freedom at each node?
>
> Don't just look at an **I-shaped** member, try it with a thrust plate connected to a jacking beam at far side; does anyone see that **Shear Center** is the culprit?

Come to think about it some more for what also happens at the girder ends;

> How is **St. Venant shear stress** calculated?
> How is the **warping shear stress** calculated?

For open-sectioned members, the accuracy of numerical-based solution to torsion-related problem via computerized means lies in part in how accurate or realistic the **BCs** were set up in the model; therefore we need to question ourselves:

Had we (not) invited both twisting and warping into the solution process? If "yes" then the resolution of chicken-and-egg arguments should come back to a few reality checks:

- <u>Ask at long last when all is done:</u>

  What is the state of cross section's planar deformation?
  *Does it not look warped? If true then the result is bogus*

- <u>Ask before even starting:</u>

  Where is the profile **Shear Center** located?
  *Is it at all or not mandated as part of the user input?*

As we "see" it, once placing focus at the support ends in terms of <u>Simple Support Case 1 in torsional sense</u> then how to "*prevent twisting about the **shear center** and yet allow warping to take hold at the girder flanges* at the same time" becomes one of the most troubled areas to clarify, thereby all kinds of misunderstanding and confusion can set in

One more thing, by checking the analytical results to see if the profile section remains plane or not, we identify the fact from a well-established certainty that the behavior of open-sectioned members under torsion is <u>entirely different</u> from that due to flexure, as we knew by heart

Therefore when pick and choose out of all six Degrees of Freedom, it would never work out for torsion from using the **simple Z-fixity** scheme, i.e. barely relaxes or restrains only the **Z**-rotation **DOF**, which could never demote or promote a flexure-specific joint into a <u>torsion simple support</u> without participation from two other important "longitudinal" effects at the *support end joints, i.e.*:

- *What actually happens at the **Shear Center's XY** coordinates and*
- *The **Z**-elongation/contraction at all locations away from **Shear Center***

In reality, employing an unsophisticated provision of **simple Z-fixity** − lacking warping influence − would have <u>only</u> altered the flexural stiffness based system, and <u>nothing more</u>

Here is why:

> Without **Shear Center and Warping** taking part in the modeling act, a specification of **simple Z-fixity** merely serves as a "user prompt" or "instruction" to the software program to either include or discard the <u>local</u> **Z**-rotational **DOF** when setting up the <u>global</u> external equilibrium equations
>
> Moreover, the numerical magnitudes of the resolved fiber stress and shear stress if based solely on flexural stiffness then, say formulated in terms only of **A**, **E**, **Q**, **I**, **L**, etc. the results would have never made any connection or distinction between warping torsion and flexural bending mathematically

*As matter of fact, once again, for problems involving **open-sectioned** linear stick-like members subjected to torsion − if not solved using closed-from formulas − there would not be **effective** ways in prescribing torsional simple support whatsoever that has anything to do with warping-related matters in <u>conventional software;</u>*

(a) <u>Unless</u> the constituent stick elements for the stiffness matrix (or flexibility matrix) were contrived from hyperbolic functions fielding cross-sectional properties either through direct user-supplied or from program-calculated terms or properties such as the **Shear Center** location offset with respect to **elastic centroid**, warping constant $\mathbf{C_w}$, **St. Venant** torsional constant **J**, shear modulus **G** and most importantly, the *torsional non-dimensional parameter* **βL**, etc. Imagine what a burden to the users preparing the input, and what a painful errand to run *especially for unsymmetrical sectioned members*

(b) Or perhaps once again, only if the software "knows" by proper programming the stark issue as mentioned and "knows" how to "take care of" the difference between a simple torsional support and a simple flexural support, etc. Imagine if true then what a convenience that is

So much on linear stick-like members' behalf, but what about using higher-ordered finite elements other than linear stick-like members, would that work better then?

> Whether of simple support or fixed support, basically similar rules and restrictions in prescribing torsional **BC** would always apply whenever considering the **warping and twisting behaviors** of open-sectioned members

Thereby as far as **CRGs** are concerned, the modeling restriction with respect to conforming to torsion behavior still holds; then "unless" on behalf of the specific type of element(s) the software does advocate explicitly the fact that the structural behavior of out-of-plane warping has been incorporated into the setup of <u>element shape function</u> − heed the word "unless."

## 5.8 Structural Boundary Conditions – As Anticipated

**Boundary Conditions** (**BC**) or **Support Conditions** being specified or assigned to a structure should complement the application-specific features called for from both analytical and practical standpoints.

In normal **A/E** engineering or design-build project progression, the structural behaviors anticipated at the **BC** were pre-conditioned ahead during the planning stage and then pre-justified in the analytical department.

To **CRGs,** the **BC** detailing work might be starting out in advance up to a point but then the rendered connection features ought to be validated following the analytical results were certified.

> The final engineered-production constitutes the most important share of the contract deliverables − whether developed into printed paper form or electronic formats − which fittingly concluded the engineering-detailing-design stage. Most of the time those activities took place way ahead of the actual structures were fabricated in shop and constructed on site

Once furnished **CRG** with the as-specified **BC**, the follow-up interest is in how would our structure behave or function in the long run; and that could only be showcased during active service.

> Whether the as-built connection performance matches the as-analyzed/modeled expectation or not can be a subtle source of concern
>
> Which if unfortunately had grown to become a major issue then it could be real bad situation; because more often than not the engineers/detailers were no longer in the picture to defend or resolve in real life

The fact to pay attention to is, the observed behavior of an as-built may, or may not at all match up with what the intent as analyzed or anticipated just so we know.

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Looking on the typical engineering-design practice from an outsider's viewpoint, one may be "curious" about what "links" between the analytical-based projection and the hope-to-be realized fulfillment.

> The link is the classic chore of "detailing"

"Detailing" sort of finishes the last leg of desktop-based structural engineering "design" activity prior to entering the shop fabrication and site construction phase. **Unless** the Responsible Number-crunching Engineer/analyst – who proposed the *sketching idea* – and the Responsible Detailer – who finalized the design/detail of the structure – happened to be the same individual, otherwise:

> Isn't it a tradition being carried on from way back and that was still (somewhat) kept on to these days, for being more practical in managing the division of responsibility and budget spent per most engineering projects, "detailing work scopes" were "implemented" by Design Specialists with or without modern aid from **AI**?
>
> Aren't these Trained Specialists more proficient in applying the graphics/design technicality with far greater attentiveness in/on the aesthetic- and/or construction- related expediency than being analytically savvy?

No matter what, there is nothing wrong in that picture and no pun intended. As expected in the end for all **CRGs**, the as-analyzed ***Engineering*** **BC** should closely match in functionality with the ***Detailing*** **BC** on drawing board/screen monitor prior to fabrication.

## 5.9 Structural Boundary Conditions – Unconditionally Detailed

Sometimes unintentionally, an end-connection intended to be simply supported may have been manipulated "somehow" into a "fixed" or "semi-rigid" support – not knowing the inherent situation could **drastically** alter the overall structural behavior and the *projected local stress pattern* – as in several situations identified as follows:

- By adding too many sets of (thick) bearing stiffeners accommodating thinner web against yielding, rupture and/or meeting end bearing requirement, etc.
- By providing tie-back elements or connection links that were relatively too rigid or too flexible along a certain degree of freedom (**DOF**) or were restrained at an *unsuitable elevation or orientation*
- By adding too many rows of seat bolts for meeting **X**-shear/**Z**-traction transfer requirement yet impeding the joint flexibility
- *Or from detailing fallacy that inadvertently introduced wind load into local longitudinal crane traction bracing component*

But what is the big deal from using fixed support or semi-rigid support?

*No big deal outwardly, herein there is no intention to denounce, discourage or encourage the practice, pros or cons, provided there is proper justification of the engineering-design intent. But if fixed end conditions are proposed on purpose then, it is important to validate their usage is "analyzed and engineered" suitably, and we should see why*

(a) A fixed support, whether purposely or inadvertently created, could bring in more harm than good in **CRG** applications.

On one hand,

It appeared to be a welcoming feature that could be more beneficial "in **non-CRG** practice" for it could (1) reduce **X**- and/or **Y**- deflection or **Z**-rotation when the deformation is measured at/near the mid-span and (2) induce much lower stress intensity (in absolute sense) compared to that based on simple supports – *normally*.

But on the other hand when **CRG** is subjected to *moving loads*,

Fixed supports could become a real bad news if the **CRG** *had not been* ***properly*** *(1) analyzed for the effects brought up from subsistence of multiple inflection points and (2) evaluated for effects due to fatigue force/stress fluctuation/reversal*; i.e. unexpected fatigue failure can take place at certain "sensitive" areas/zones despite the nominal stress intensity was at much lower level than the material yield point.

Why is *moving load* a curse to fix-ended (**CRG** or **non-CRG)** members?

Consider what we don't want happened to **CRG**, "unexpected fatigue failure" is the key happening.

When **CRG** is subjected to lateral and vertical (**X** or **Y**) load and associated **P-δ** effects as usual:

For example, to *single-spanned fix-ended* member, there could/would be **multiple** *inflection points* tracking along the elastic curve.

Consider viewing a typical flexure bending moment diagram, *Inflection point* is the point of zero moment; where the value of bending moment approaching from the adjoining nodes would progressively dwindle to zero and then reverses the numerical sign once crossed the demarcation point – the *inflection point*.

*Thus in a very big way when considering both bending moment and flexure shear, the <u>inflection point</u> together with its <u>vicinity</u> are the most probable location where (1) tensile stress can experience fluctuation and (2) shear stress can experience reversal.*

The location of inflection points would have stayed at a fixed node so long as the loads remain (1) stationary and (2) being applied at location having fixed node coordinates.

The issue is with moving loads;

It purports a not-so-palpable occurrence in that the inflection point becomes floating in phases simply to keep in pace and to a point catch up with the moving action as if following along with the wheel movement but, *only so to certain extent*.

In turn out not all but a good portion of nodes along the member's span might *experience bending moment sign changing and/or shear reversal* – and for certain it gets more complicated when (**P-δ**) torsion is also joining in the conversation, we meant the action.

Driven by the crane operator's action, every wheel takes its own turn to go back and forth stepping in doing its own thing and then leaving off away from the girder span; imaging what can multiple wheels do.

Then what can happen in a continuously spanned situation?

More pronounced case in point of which is, if a member carried on its framing outreaching further into the adjacent span then, its elastic curve may display multiple inflection points unveiling rather diverse features depending on:

- In one single pass, (1) how many wheels are participating in the action, (2) established wheel spacing and (3) where are the wheel loads were positioned:

  What needed to be kept track of becomes rather unpredictable. Were all wheels congregating at the near side span only or all at the far side span only? Or is it some here and some there in variable series spreading over both spans?

- How the member end(s) were engineered and detailed as documented in black and white?
- How had the as-detailed/as-built support(s) actually behaved in service?

*Thanks to moving loads for being so hyper-dynamic in nature,*

*It can cause the whereabouts of the inflection point to <u>float or fluctuate</u>, becoming irregular as they are relocating with respect to the dynamic timing change.*

*To us, Engineers, the complexity incurred from the chaotic global load responses and the associated local stress patterns would make it impractical to "guess" or "follow" if the data domains were relying on traditional fix-sized arrays or flat-file database setup, i.e. which is not primed for dealing with volumetric numerical mess through robust data management scheme*.

Consider the all too dynamic happening to each and every **Z**-thread along the longitudinal stretch, owing to the constant charging action brought from ***X****-bending,* ***Y****-bending and* ***Z****-twisting/warping*, then as consequential as rightfully so, there a total of three distinctive effects are to be remarked.

The implication:

Like it or not, we have three distinctive elastic curves to content with, and each one has its unique configuration subsisted in the **3D** space dictated mostly by (1) the section properties (**E**, **G**, $\mathbf{I'_x}$, $\mathbf{I'_y}$, $\mathbf{Q'_x}$, $\mathbf{Q'_y}$, $\boldsymbol{\omega_n}$, $\mathbf{S_w}$, **J** and $\mathbf{C_w}$) and (2) notably, the support Boundary Conditions (**BC)**.

Plotting out all the elastic curves through influence line analysis is the only way whereby one can't help but to recognize there are differences between bending and warping.

Potential problem:

*Unmistakably, the* ***BCs*** *of interest at this point are those as-detailed/as-built as is, not the ones as-analyzed or as-assumed*.

By that it meant; the tallying of how many inflection points there were actively present becomes the center of attention.

With an all-inclusive intent to cover all probable aspects, the number of inflection points in each of the three elastic curves at any instant and for all instances associated with the applicable (moving) load pattern(s) must be accurately accounted for – only through proper calculation.

Besides that, the (unpredictable) spin-off from applicable loads acting in reverse sense needs to be included in the stress combination per specified load combination, too.

On a side note, can anyone not see the data tracking issue looming all over yet? But then after all, there seems not much to worry if we do know where the loads are applied at, correct?

Fortunately and unfortunately so, the answer is a tiny bit of Yes but a good number of Nos.

The generic problem from using **continuous spans** is in what it takes to accommodate every case based on the crane wheel marching order – that could engage a single wheel or a group of wheels.

It would be of no big deal provided there already stockpiled in our treasure chest an all-inclusive data management schema so prescribed (1) to keep track of global load response linked to every probable wheel load pattern and (2) that in accordance enabled to account for variation of all local stresses combining flexure and torsion effects induced thereof.

Otherwise for **continuous spans** we would have no control of where the loads are landed because all wheels can be driven from one span to the next span back and forth all under Crane Operator(s) control; in turn we might not know which elastic curve is flexing incessantly and which one does sporadically, or is it all or nothing.

On (global load responses and) internal stresses:

- How may "fixed ends" condition hurt a structure through repeatedly flexing of the elastic curve(s) alone is not that difficult to "picture" if only by pure imagination, but in reality it is very difficult to put a tap on any specifics

Why? The issue is; the member would experience series of "switching over" from one inflection point to two inflection points, and then flipping back to one point, etc. time and again, so given the situation, keeping track of each and every state of stress fluctuation and reversal for fatigue assessment becomes a "little bit of" challenge if doing the chore without a proper back-end setup

- The immediate location of these inflection points "floats" constantly from one **Z**-spot to another different **Z**-spot(s), the multitude of wheels moving across <u>multiple spans</u> could nurture a pressing data management challenge

  How does the inflection point(s) "float" – from one span to next span – would depend on which axis (**X** or **Y**) is in the spotlight, in other words the instantaneous *flexure bending moment diagrams* for **X** and **Y** may not share common configuration with the (1) *flexure shear diagrams* and the (2) *warping-based bi-moment diagram* thus making any "unstructured" guess work completely obsolete

- Imagine some of the local region(s) of the elastic curve may bow up one minute or second, and then cave down the next minute or second, or lean sideway towards the left for a flashing moment and then lean towards the right a second later, resulting into an **XZ**-based convex shape from **X**-bending at one second, then changing into a **YZ**-based concave shape for **Y**-bending, etc.

  Just from considering flexural behavior alone, the level of difficulty in the handling of such randomized numerical twists-and-turns quickly multiplies if also taking into account the load sense reversal effect. Then on top of that, warping would join in playing with its own temperament in this regard that needed be dealt with, too

Sometimes what deemed normal by common layman-styled wisdom could be brought down into a state of disbelieving if not astonished once we jump into the "details" in view of such a complex load-response relationship;

Problem is do we really know what details to look for.

Normally,
To the terminal tip of a selected top flange at certain **Z**-coordinate, it would have stayed in compression considering flexural bending about the **X** axis due to the relatively heavy **Py** load alone, *but,*

*Depending on (1) where the local **X/Y** node is located with respect to the **elastic centroid** and (2) how far off the **shear center** is from the **elastic centroid**,*

The same flange tip could experience compression in one moment then tension the next moment when combining the bending about X with (1) ± bending about **Y** and (2) ± warping stress, (both were reversible by themselves)

And thus for certain load combination event, the combined stress (**SRSS**) in a more general sense could be straight positive or negative, or in the middle of becoming **X**-reversal or **Y**-reversal, or whatever it may turn out to be, and then we need to ask ourselves:

- Would there be cyclic tensile/compressive stress fluctuation?
- What about local wheel bending in the top flange?
- What about flange/web post-buckling effectiveness?
- What about shear reversal? …

Therefore from such an all-out disarray in **3D** state of flexing, shearing, twisting and warping, etc., it is <u>not prudent</u> to (1) single out specific zone(s) within the girder span as the sole center of attention − without considering other possibilities such as inclusion of some critical flange nodes or other **XY** node(s) unseen, or (2) so boldly to assume that certain zone(s)/node(s) could be exempted from "detailed" fatigue risk evaluation; no matter done it wrongly or correctly, but, for fixed supports, we have to look for every probability

The lesson is, do not skip anything based on blind trust of "incomprehensive" or "flawed" traditional wisdom or assumption alone; for somewhere there could be a hot rail connection in the web or a rail clip connection in the flange, all these similar/dissimilar features could be populated all over in various zones, then the worst that could happen is, many of these connections and vicinities nearby were handicapped with extremely low allowable fatigue strength.

(b) **Semi-rigid support** is rarely a comfortable "thing" to enter into engineer's everyday conversation, especially the semi-rigid torsion support. And yet semi-rigid torsion support can be allegedly made to "exist" depending on how the girder ends were "actually" performing-behaving on site, albeit semi-rigid torsion support was not intended to be brought into the being intentionally.

Since the "thing" showed up here for attention and just so we recognize the reality, semi-rigid support has two different types; flexure and torsion.

For sake of making arguments, this infrequently talked-about support type – being brought up herein in all fairness to all comers to decide what to do about it – may or may not be a serious technical issue on selected conditions.

Semi-rigid support is never a good idea for the unqualified girders to consume, we meant for engineers to content with in technical sense. It would be **OK** provided that we do know how to deal with the technical consequence from semi-rigid support in the analysis, and justify it by "hard numbers" throughout the design process, again, how to do it correctly is the problem

Otherwise pure talking on the subject or blessing the design without hard number is Never Good for a fact. Anyhow, semi-rigid support may not be that difficult to handle if considering only the flexural effects without considering effect from moving loads.

But on torsion's behalf, semi-rigid support is definitely a big controversial "can of worms" that should please no one if opened up for debate. Or take the middle ground, do it twice, once as fixed once as simply supported?

## 5.10 Structural Boundary Conditions – Bad Examples

Of all discussions on **Boundary Conditions** thus far, the notion on what to watch out for *before and after* analyzing a **CRG** might not sound that alarming just yet, but it deserves to be repeated once again, albeit most Structural Engineers already knew what to do by heart:

- Always verify if the as-input end condition closely matches the actual (or as proposed) girder anchorage and tieback features

- Always verify if the overall **structural behavior** and the long-term **performance** on site closely match with what was anticipated in design

During typical girder project development – irrespective of what **Boundary Conditions** (**BC**) were (to be) specified – Engineers and Detailers at the end of the day (or came to a good stopping point) needed to consult with each other, making sure qualitatively the as-analyzed and as-designed intents were not end up with (fancy or copy-and-pasted) details lacking needed strength to take on the effects as result of incessant interfacing between flexure and torsion.

Several situations exemplified as follows should merit our close attention, even though we may be repeating some of the forewarnings already been addressed:

- **Fixed End Condition** is not recommended for CRG application. However, if a connection was intended to function as "fixed" then it should be respected as truly fixed through proper engineering evaluation and detailing

    A typical **CRG** engineering-design qualification process should always include both fatigue and non-fatigue assessments, enough said – the art is in how not to lose touch with the influence from "floating" inflection points subsist in elastic curves owing to "unpredictable nature" of moving loads; *otherwise don't use fixed end supports*

  Dare to ask again, what seemed to be the problem with fixed supports that we needed to pay much attention to?

    As in the past (and perhaps in present days, too,) some **single-spanned CRGs** were purposely (1) made continuous over multiple spans or (2) for the worst propped with knee brace(s) from underneath a simple-spanned member into a pseudo-multiple span

        All those approaches as mentioned was done on purpose with a modest engineering aim to curtail flexural/torsional deflection/rotation at intermediate nodes – a commendable practice that should work well in most **non-CRG** applications, but, there is "serious" downside that comes with it given what were covered earlier on

    Then in those **multiple-spanned** cases, instead of analyzing/modeling the given scenario as continuous members, many Engineers would "cleverly" exploit an oversimplified scheme in that each discrete span was *mathematically* cut off at the continuous ends, and thus the cutoff was analyzed duly as if being fixed at both ends on its own merit − thinking it was conservative by doing so (debatable again, perhaps true even to most **non-CRG** applications)

  What really happened to the members with fixed ends is, whatever benefit gained from cutting the continuous span into **separately detached span** could be superficial

    Any (fictitious) gain in savings of material or simplification in detailing and fabrication efforts upfront could nonetheless be met with material fatigue at long last, likely *due to underestimated, wrongly estimated or none estimated on the span-wide magnitudes of tensile stress fluctuations and/or shear reversals*

  At this point many engineers wonder; why propping knee brace(s) from underneath a simple-spanned member is the worst idea?

    Basically by adding diagonal knee braces with one at each end, when viewed from the side or looked in elevation; the full-span length has been crafted into a continuously connected support system having three smaller spans

    That seemed like a good idea to start, except:

    - It works for symmetrical sectioned members. To unsymmetrical sections, the bracing must be in line with **shear center** location

- Or it works for non-**CRG** application observing “simple bending” mandate, otherwise shear center will do its magic and we all understood what that is

- Or for unsymmetrical sections and **CRG** application, it only works when the system is supplemented with lateral bracing installed at the matching connection point with the diagonal bracings.

  What happens if lacking lateral bracing? For vertical loads, it is on a 3-span system; but for lateral load and **P-δ** effects, it is not 3-span but a full span. One way or another, it would be a nightmarish challenge to the analyst, just think about what to do if “moment distribution method” came to mind

Still, would anyone be in favor of using knee bracings?

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Take an off-subject and tie it in with some unhandy interest of **CRGs**;

Remember the good old flexure moment distribution method? The subject is brought up for argument’s sake

Solution to modern structural engineering problems relied so much on fast-track automation these days that many Structural Engineers thought practicing the classic moment distribution manually has become the thing of the past – yes, maybe

But just in case automation tool is not present or is not properly set up for the on-the-spot analytical intent, moment distribution might still be one of the simple techniques used to solve simple problems involving two- or three- spanned continuous beams

That seemed “outdated” to some but might not be that outdated and really there is nothing wrong, especially for “quick solution” toward meeting **non-CRG** design intent with non-fatigue implication based purely on *flexure influence*. Yet unfortunately,

> **CRG** does not “fit” to take advantage of this so-called “quick/slow solution” for obvious reasons – moving load condition for one

One of the other problems besides moving load is, for typical **CRG** application, in order to maintain the handling of stress reversal matter and keeping it under control, one must “play” with both flexure and torsion prudently – recognizing which one(s) of those constituent stresses are reversible in sign and what doesn’t – as we had already learned

But no matter what we choose to use, it is much easier in dealing with simply supported members than fixed end members, plainly because they deform in single curvature under simple flexural behaviors

Now here it come the touchy issue, albeit we have “quick solution” in mind provided there is no concern about moving load for time being. It is never easy (or impossible) to make it truly work out manually in practical term for **CRG** (of open-section) under torsion involving continuous spans;

> All because in addition to performing flexure-based moment distribution, there is also the need of execution of a more advanced bi-moment-based moment distribution procedure, only so doing we were able to properly deal with “warping normal stress” prior to bringing that along with flexural bending stress for fatigue qualification’s sake

*Bi-moment distribution* may or may not be familiar to many Structural Engineers, the fact is, digging into it could be another “technical can of worms” (and that is another special topic on its own)

Yet after all, flexural moment distribution (*moment of inertia based*) and bi-moment distribution (*warping constant based*) are the two monstrous structural species very seldom had they met each other in the same design instance in traditional applications, but they sure would in all open-sectioned **CRG** of continuous spans

Again, for argument’s sake:

Even if bi-moment distribution was perfectly/properly executed – if not done for showing-off purpose – it is still not enough to merit the bragging right because what it accomplished has only taken care of the warping normal stress at best, but what about the **St. Venant** shear and warping torsional shear? Without that then how can we qualify the structure against *shear fatigue*?

| |
|---|
| The topic/issue of (1) multiple inflection points and (2) bi-moment moment distribution being brought up hereinbefore is only to make a point and to demonstrate how difficult that is and what it takes to qualify the adequacy of multi-spanned **CRG** |

- By the same token, **end conditions** meant to act as *torsional simply supported* should truly function and behave as designed and as constructed as well

  Easier said in some cases:

  Every Newly-minted Crane Runway Girder from an engineer’s desktop can be “assumed” and be “expected” to function well in working order as if it’s done flawlessly − on paper/screen as analyzed as we would think − with all decent engineering intents;

  But performance-wise, progressively at some point in time during active service, and the girder could either be all good and trouble free or else fall short of expectation if certain components were not properly *engineered and detailed* to begin with, enough said, again

  Not so easier said in other cases:

  <u>Take some of the **CRGs** having relatively (1) deeper depth and (2) longer span than the adjacent members as a classic example:</u>

  When the end(s) of a <u>longer/deeper</u> girder met with <u>shorter/shallower</u> member(s) came in from an adjacent span, very often these deeper members were purposely notched (deeply coped) at the top to provide seating support for the shallower girder − as seen as in some of the (popular) detailing connections *at the interface where these two girders meet* – by doing so the deeper girder now becomes a **variable sectioned member**

  Examine the connection detail with such arrangement and see what happens;

  It **appeared** (as if) the junction-interface was so well designed – by the look – for being so fair and square given such a <u>clever-looking appearance</u> by way of a well-fitted bench/seat plate being welded over the cut-out web of the deeper girder, which in turn was set on its own base/cap plate over the crane column

  So far so good if judging merely, again, by the look, “the seating arrangement” may work fine (<u>as if</u> free from any serious side effects) for the shallower member – but the hardest part is, *if only analyzed/designed/qualified properly*

At any rate, what as-envisioned in the olden days through what as-designed is really not that rosy; in fact, if we looked more closely,

In order for the deeper girder to carry the burden passed on from the shallower girder, the connection as detailed had "morphed" the "deeply-coped" girder seat(s) into somewhat semi-rigid or semi-flexible end(s) − **especially against lateral movemen**t – that inadvertently brought down the intended effectiveness to an unknown degree with (substantially) reduced section rigidity or stiffness owing to the notching

As already understood, the connection as detailed would have functioned well *if only we know "how" to analyze and design it properly*; but what revealed in the Mills seemed to have proven the opposite is true and it doesn't take much to "see" the true picture

"The localized seat" obviously lacks the required continuity in strength to fend off the (direct and indirect) beatings combining assault from loads on both the deeper and shallower girders in an erratic **3-D** nature

The bottom line, the "seat" must endure, for the least, the random attacks of stress reversal due to the back-and-forth trolley swaying along **X**-direction among other applicable sources

The serious ramification grossed from the shoddy "engineering" practice becomes an artificial inadequacy that is also a "purposely" fabricated "geometric imperfection"

It's one thing that the deeper girder as a whole has been shortchanged into a handicapped situation with severely reduced strength due to the notched corner

But it's another thing for the "handicapped seat" that must at all times perform double duties not only taking the **3-D** loads of its own share but also from the inevitable interaction with the shallower girder's influence as extra burden. In a way the deeper girder can still feel the pain even though the crane is working on the shorter/shallower girder

It should be of little doubt that some of the "bare" base metals at certain critical hot spot(s) in the vicinity of the "seat" – *in particular the base metal/weld immediately under the seat* − are prone to meet their fate much sooner if not properly qualified and/or reinforced

When subjected to repeated loading-unloading rounds of forces − actions-reactions pointing along **random** senses in the **3D** space − from which the ever so unpredictable structural behaviors and the irregular stress pattern inherent in (and near) the detail would make it much more vulnerable and thus prone to chronic cracking in the base metal at the seat region − a **RUP** circumstances found very common in ill-fated facilities with this kind of clever-looking-but-not-so-clever seat arrangement

Many unsuspected Engineers may wonder why a good looking and somewhat robust connection like that would develop cracks

The puzzle would be, even though the seat "seemed" to be under tremendous amount of compression along the **Y**-axis, and so more than likely, metal fatigue owing to tensile stress fluctuation should not occur and should have no control at all

Plus the verity on their firm believing, there "seemed" so much friction resistance in command thus the seat should be free of tensile stress fluctuation; *and yes it sounds good, and only sounds so true*, but think about what can happen in reality

The fact is, unless the state of stress in the region had been accurately analyzed and evaluated against all probable failure modes, otherwise no one can really assume or predict what could (not) happen to the seat one way or the other

And most of all, has anyone gave it a serious thought, what about the unpredictable (and the unaccountable) “shear stress reversal” and “shear fatigue?”

Consider the *combination of bad detailing with bad luck*,

Has anyone ever suspected if there could be non-**CRG**-related problems looming − over and under

Things happen, and that could come from unsuspected sources such as *settlement of the foundation*, out-of-square crane, or other critical issues that no one had ever pay due respect to or factored in the design consideration? Evidently if not rarely, but there could be unexpected non-load-related situations impending more damages than that from simple stress-and-strain matter

Even though by a long shot, many Strong-willed Engineers may not agree right away with the fact that there could be so much manmade sorrow and grief that can bring to so many **CRGs** from bad detailing, perhaps the best part to them but the worst part in reality is acting ignorant. Anyhow, but we must give a reasonable answer to the next question; pick a bad spot on any **CRG** just for talking purpose:

*If following repairs to no avail after so many repairs after series of prior repairs and it still cracks then, isn’t it about time to revamp the connection-detailing concept or look elsewhere nearby to “see” whether the foundation, column tie-back or the roof diaphragm for clues, or would we be better off replacing the girders with a different detail?*

Unaware of the fact or not convinced that the notched girder is a real bad idea but then someone decided to prove it works just fine through analytical means, so what about using a “local cutout” finite element model, as some engineers of the “nervy engineering outfits” had chosen to do exactly that

It is interesting to “see” how it works, just imagine –

Before making a model, we have to think about (1) what could happen when looking directly over the “seat” region and its surrounding carefully and (2) what load to use when considering those ever changing in the “spread” of crane wheels marching on top of both the deeper and shallower girders simultaneously (or could be on either one girder but only one at a time)

Then imagine further on –

First of all, is the unpredictable state of “wheel spread” that regularly causes commotions in all **X/Y/Z** directions a biggest concern?

Secondly, for all practical reasons, there is basically **no justifiable way** to justify “mathematically” if it could truly “work” − for simplicity’s sake − by cutting out the deeply-coped end piece of a **CRG**, to (1) make a local-scoped “analytical model” out of it for finite element analysis and to (2) analyze for such erratic load pattern, all seemed (un)reasonable

Will it work after all that effort spent like some quick-witted engineers had done before? Politely said, it (probably) won’t; harshly speaking, it will fail without a doubt

Here it goes again with the reasons why it doesn't work:

Before we could make a "run" of that local model, we must consolidate a collection of realistic **moving** load cases from all **probable** wheel marching patterns

Then in applying that load chain to a localized standalone *surrogate* model − which is no more than a "carved-out" end **notch** piece

Next in no time we run into a major problem: Treating the notch piece as if it were a "sub-assemblage" or "substructure" requires idealization of **boundary conditions** properly appeasing the interfaces:

(a) At one end with the "much deeper parent structure," from which the piece was severed from and

(b) At the other end with its immediate interface with the shallower girder sitting directly above

And supposing such a "questionable model" was technically justifiable and was flawlessly constructed, but, once done with the analysis, how could we rationalize or validate the state of stress and the deformation patterns including warping?

… Besides all that, here are more questions:

- How to take charge of the displacement and/or force **compatibilities** at boundary nodes with the parent structure?
- For both the deeper girder and the shallower girder, how to evaluate the effect from shear fatigue stress reversal? Or even how to correctly calculate the flexure-based and torsion-based shear stresses?
- Was warping involved in the shape function of the finite elements being modeled with? And which software is capable of such feat?
- From the result, can we correlate that into valid reasons explaining why is there a crack here, or no crack there? Shall we be more specific or more serious? …

- There is another poorly designed **end condition** example similar to the last one that also involves long-span and deep girders having a truncated bottom at the supports instead of being notched at the top

  The worst of its kind is (1) making a "cut" from the bottom flange straight up into the web at 90-degree then (2) turning longitudinally into a horizontal run without a sizable rounded fillet or transitional bevel

  This seemed not as bad as the girder coped from the top for seating, but sure enough it would be of same fate if not analyzed and detailed (or reinforced) properly

## 5.11 Structural Boundary Conditions – Lessons Learned

One of the important lessons learned (*or to some not yet learned*) from what as elaborated up to this point is, never take those seemingly clever-looking but not-so-easily qualified details/connections too lightly; i.e. don't fall for the appearance only to find out there is no way to justify the adequacy against metal fatigue.

As demonstrated in §5.10, deep coping the top flange of a deeper girder to provide seating for the shallower girder is a *very good example of very bad design*

Main reason is, when subjected to application of **3D**-natured moving wheel loads – *every load point passing through along on the top flange over/near the "seat" region can bring in **X**, **Y** and **Z** loads concurrently* − the seating region would be in a state full of twists and turns as result of the unpredictable structural responses and behaviors subsisted in the connected part(s) and the surrounding areas, too

Nonetheless, to some of those chronic naysayers, all that as explained to them so far seems to defy their *normal* intellectual perception. What struggled through their mind seemed to habitually fall under their *normal* **non-CRG** way of understanding of structural behavior, which ruled mostly by their own *normal* brand of science behind simple-bending.

Anyhow, was anyone not able to identify who are the most damaging perpetrators to **CRG**?

*Is it fatigue, torsion, (none of those) or both?*

Sounded like repeating a disrespectful tactic of questioning, but we all should be able to answer it by now.

Granted so, we should "know" the hidden/unseen (or obvious) perpetrators as they are in real existence, but in the end, what should we do with those aforesaid not-so-ideally analyzed/detailed/qualified connections? Even though after learning some (not all) of the *why* and *what*, but without learning the proper *how* part, it won't help any if letting the issue off the hook without a better way out; then let's see:

First of all;

At this stage being this far into the subject; it shouldn't take much for anyone to have fully vetted "engineering 20-20 eyesight" to "see" clearly what damage that torsion and fatigue can cause if we, the engineers, did not take the full responsibility to deal with the issue. To say we have done it, then show and tell the "how" part in the calculation.

It is that simple, when in doubt, look for clues and answers. Then from a Design Reviewer's perspective while taking on the matter, *just look for any "documented" clues in any of the **CRG** engineering calculation and "see" what is missing*.

Not in a mocking buzz for more boredom, but some of us need to be called attention to the somewhat gloomy if not so depressing <u>realities</u> in the mainstream **CRG** design tradition:

- The **deep-seated underestimation** in the magnitude of (potential) damage that can be caused by using bad tradition in engineering practice is a fact. It needed be reiterated for good reason; for many are unaware of "collective hazards" can be led from (1) failing to take proper care of torsion (being a huge headache to engineers) and (2) ignoring the must-do calculation on required strength against material fatigue (being the major nuisance to the structural entity)

- There is a **serious disconnect** in telling what difference is in between how **CRG** works on paper and how it has to put up with once being put in active service

  And thus no surprise, many had no idea of how do some of those critical connections as-detailed actually "suffer" in service. Even though the design was clearly flawed but then still in a hurry to get it signed off anyways as if the design of girders/connections were faultless

**<u>Misconception</u>**:

Much as we were so accustomed to "seeing" how flexure has been established all along – throughout history – to be so prevalent in common structural engineering relevance, driven by such a formidable historical norm that no wonder no question is raised against "not seeing" the importance that torsion also merits its own share of relevance in **CRG** applications

Treating incompetently or wrongly to torsion matters has been mistakenly **OK'd** by the mainstream, which may involve giving inadequate attention it deserved and by furnishing no defense against its all-negative stimulation to the structure

By allowing such undeserved notion to go wild on the loose, the unsuspected ones tend to "rationalize" the misconception and then take the self-approved measure that, torsion is no bigger deal than any "other" allotment of structural effects just like flexure does, but is it that so simple?

**Underestimation**:

Accordingly, for conventional **non-CRG** structures subjected to conventional (centroid-based) loads that do not see torsion eye to eye in their path, therefore once given a real-life **CRG** to qualify, many unfazed individuals underestimated the threat that torsion can cause, so they would either relegate the chore to dealing exclusively with flexural effect only or else get by in mistake from using inadequately amount of torsion, or even resort to taking shortcut via **Flexure Analogy**

Take cap-channeled girder as a bad example, customarily the lateral thrust loads were shoveled entirely over for the channel to endure so as if there is no such thing as torsion – the wishful thinking prevalent in this case is, everything works as if the structure is free from torsion's threat

And yet even so keeping torsion at arm's length, but the structure's in-service longevity still needs to be justified so long as the "design mandate against metal fatigue" is part of the objective, torsion or no torsion

Secondly,

It doesn't take much to "see" rather clearly, in all **CRG** applications, torsion would not exist without flexure's presence – in fact they coexist at all time.

In a very big way to unsymmetrical sectioned **CRG** structures, (1) torsion is one of the triggering sources of trouble that is not going away whether we like it or not once we find out where the true **shear center** is located whereas (2) fatigue is the distressful result that the structure always bears the potential to see it play out whether we see it or not in our calculation

As **CRG** is exposed to (1) both centroid-based and shear center-based loads and (2) the mutually "see-sawing" in-phase/out-of-phase effect from selected loads acting in reversible senses,

For **CRG** to enjoy the best possible outcome in service, only if we furnished with sufficient strength backed by proper calculation then, the structure may or may never experience material yielding or metal fatigue in real life − at least in the beginning before things were heated up – all depending on how the structure was actually **loaded** in service and **qualified** on the desktop

But once the structure went through a more intense beating in service, situation could change in the long (short) haul depending on how it could sustain cycles/passes of phasing in and phasing out of each individual effect's partaking

Fatigue and torsion, each entity can post either "no threat" or post many <u>different forms</u> of "threats" there applicable to each and every **CRG** in a very unique way; on various occasions, some of the selective structural components may experience through only one form of threat, all forms or none at all

For structures of all ranks, on the dealing with whichever culprit hidden or exposed, the proper provision in safeguarding against <u>any one specific threat at a time</u> seemed technically matured enough for practical purpose *provided there is no concern of fatigue and there is no load inducing torsion*, by wishful thinking, of course

But for **CRGs** as many engineers would not know, the "would-be transgression" and the "imminent troubles" that both fatigue and torsion could bring onto the structure were much different from what brought onto other types of (**non-CRG**) structures

> Learning and understanding what terminology to call to mind in philosophical sense is much easier than to deliver a true deed in how to be successful in **CRG** engineering;
>
> Why? Because the discussions on how much different there is between **CRG** and **non-CRG** and the reason why they are different remained markedly small talk (or no talk) in many mainstream design/engineering quarters. This could be a worldwide problem, or already has been
>
> And even if someone does touch on the subject with respect to qualification of the design of **CRG**, it seems to have stayed mostly in "theory" or tucked in the "design criteria" with not so much evidence shown in actual "calculation" serving as official endorsement to structural engineering-design quality
>
> Then even in taking care of matters on fatigue damage to **CRG**, many advocates seemed to prefer talking loud than taking action. To those in the learning to do it right or started making calculation for the cause, they seemed to focus more on *influence from tension at bottom flange by **MC/I** than that from shear taken place elsewhere* … see the problem of lacking participation from shear yet?

Now for argument's sake once again, those "imperfect" connection details brought up hereinbefore (and many more not yet mentioned) may survive in other Industries, but it really is a toss-up for **Crane Runway Girders** in the Mills for obvious reasons.

> Without "detailed calculation" as legitimate backup, those Engineers and/or Detailers would have felt so marvelous at pioneering or on replicating the "ingenious details" being so unconditionally honored in the good old days − back then and even in modern days, too − but they should be pretty dreadful as we speak up to this moment in time once seeing their "masterpiece" one after another got hammered into **RUPs** of late if not sooner, especially on those girders mostly older than half a century or the ones already in their mid-life crisis

Lessons, if not learned yet then it is better to pre-learn ahead of time and it's never too late to know:

> *On any **connection details** or any Engineered **Boundary Conditions**,*
>
> *If for which (1) the state of stress or the structural behavior of that connection and (2) the side effects in the vicinity of certain key components (1) appeared questionable to our peer reviewers or even to ourselves, or (2) whatever that became an ever challenging nuisance to any engineer as to justifying the structural adequacy, especially in "fatigue" and "torsion" senses, then probably it's not a good idea using them in practice after all*

While we're at it, there should be no better topics to finish off with the **Boundary Conditions** matter than expanding our state of mind further into the next subject: **Finite Element Analysis**.

Many Engineers and technical gurus <u>may or may not</u> have tested their hands on it, but some firmly believed that the whole business of **CRG** structural analysis along with the resolution to those pesky fatigue and torsion issues could be managed through **Finite Element Method**, really? But for **unsymmetrical sections**, is it easier said than done?

## 5.12 Finite Element Analysis – Users Beware

As of this writing recalling **AISC Specification** on **unsymmetrical sections**, for which the Commentary (as of this writing) suggested that *the stress distribution and/or the elastic buckling stress be determined from … or **finite element analyses***.

Depending on what drives and whatnot, usage of ***Finite Element Analyses*** should be <u>nothing wrong</u> for general purposes.

> But, is it true, if by applying the method to real life (vs. imaginary) **Unsymmetrical Sectioned CRGs** then, can we comfortably "assume" the resulting *stress distribution* – besides *elastic bucking stress* – would "correctly" reflect how these members actually "feel" in service?

How to make calculation of the so-called *elastic bucking stress* – only if it's practicable – and understanding what that really mean are what many engineers wanted to find out. Is it related to *lateral torsional buckling* or a different mode of buckling?

Other then that, wouldn't anyone be more motivated to know more of what might happen to the inner body parts of a typical unsymmetrical sectioned **CRG** under load?

> Anyhow, many practitioners may raise a very good question here before long; why bringing up *lateral torsional buckling* here for? Why? Isn't *lateral torsional buckling* a phenomenon owing to <u>strong axis bending</u> for loads passing through **shear center**?
>
> Even if we choose to talk about flexural buckling as a different phenomenon, that incident is caused by axial load that may or may not pass through **shear center**. Regardless, it can never take place because **CRG** is axially too strong to fail under this mode
>
> The interesting thing in real life is, loads on a **Crane Runway Girder** are applied at the top of the rail; which had very little (or no) chance to pass though **shear center**, even for symmetrical sectioned girders
>
> More interestingly, if we step into a Mill to see the real thing, we see girder fails not by buckling but by material fatigue. Then the mystery comes down to, what kind of buckling stress we can use finite element method to determine?
>
> A more serious concern is, how many among us had tried their hands <u>successfully</u> on using finite element method to analyze structures having **non-I-shaped** unsymmetrical section and feel **OK** without bringing up attention to:
>
> - How to deal with the potential of local buckling that is prone to come about to certain cross section components with relatively higher aspect ratios than that of compact elements and
>
> - What to do when the orientation of applied load resultant does not pass through the **shear center** of a cross section?
>
> Then before going any further, once again to those self-proclaimed experts in the very topic of **Unsymmetrical Sectioned Crane Runways Girders**, do they realize – as mentioned time and

again – that the applied load resultants (*we are talking about resultants in vector form, not the individual load components*) had no chance to pass through the **Shear Center** in real life?

Assuming **AISC's** suggestion does make perfect sense just for fair argument's sake, and even so, one may inquire for educational purpose:

- What is the distribution of <u>non-buckling-related stresses</u> can be identified?
- What type of <u>buckling stress</u> can be identified or determined? and
- What do these two categories of stress mean to qualify the design of **CRGs**?
- What about fatigue?

Anyhow, there is huge difference in using **Finite Element Method** for purpose of solving homework problems and solving real life **Crane Runway Girder-related** problems.

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**Finite Element Analysis** (**FEA**) facilitates numerical-based solution by "running" computer software developed expressly on specific cause, which may include applications correlated with mechanical engineering, electrical engineering, chemical engineering and structural engineering, etc.

In all relevance, we as users (or developers) communicate with **FEA** tool – so it could "do" its job for us and "give" us the answer – through Input-Output (**I/O**) interface.

As a further matter we should recognize, our application is subjected to:

- **Constraints** imposed by combination of hardware platform, operating system, supporting software language and the developed software tool, etc. and

- **Limitations** ruled by the correctness of our own problem definition and whether the end results befit the intended purpose of what we were setting out for, etc.

So long as the **I/O** amenities were administered appropriately and being followed fittingly on software developer's (bot ours) term, there "seemed" <u>nothing wrong</u> with using **FEA** on selective structural engineering applications for pure analytical interests or that being part of the more complex process towards meeting general design intents.

However, <u>to **CRGs**</u>, electing **FEA** as if an incontestable tool as directed in **AISC Section F12** for analyzing structural members involving ***unsymmetrical section*** under ***torsion influence*** could either be a wise choice – yet to be proven beyond academic **R&D** sense involving moving loads – or be a very unwise choice (and we shall see in **practical** sense) to certain degrees

When it's all said and done, and regardless to however we arrived at the conclusion, the true usefulness of **FEA** for practical purpose would highly depend on:

- What are our anticipated "benefits" from using **FEA** as problem-solving tool and
- How "competently" can we manage the **I/O** toward reaching our design qualification goals

As productive instrument through "running" **Finite Element Analysis**, we could harvest a whole lot of "intelligence" rather conveniently from behind a wealth of **output** (information) yet with relative little **input** effort once we get it going.

No matter if the analysis was for obtaining *stress distribution* or *elastic buckling stress*, etc. as **AISC** had suggested, at this point, it appeared the practice is more satisfactory for most other **non-CRG** applications for several reasons to be described later.

In dealing with **Crane Runway Girders** subjected to moving loads – *provided that the basic **I/O** data-structuring plan (1) was adopted suitably for handling influence line analysis and (2) the routine was executed flawlessly* – we should anticipate a swamp of "underdone" stress-related information in wake of wheel load combination process, and that could inevitably create an information overload crisis whether we wanted it to happen or not.

> Bringing up such an "advanced caution" has a good reason;
>
> It seemed anti-productive, except in a more subtle way, the "advanced caution" should prime up the awareness of a "major numerical annoyance" that would follow. Why raising that concern? Because the qualification of any structural entity to be **fatigue-proof** or be **fatigue-resistant** requires much more than obtaining raw − *not-fatigue-assessment-readied* − stresses

As far as **CRG Engineering** is concerned, "Analysis" and "Design" are two distinct but closely related sibling activities − as required to accomplish a cascading top-down structural qualification session – and in normal flow of **CRG Engineering** progression, these "two" activities had to be implemented through "two" discrete modulate conceptions and yet sharing the same robust *data normalization-data linking* scheme.

> Dealing with **Crane Runway Girders** as to consolidating the run-of-the-mill information into serviceable data depositories catering to both fatigue and non-fatigue assessment purposes befits a unique **CRG**-flavored data administrative undertaking
>
> In that each numerical element in the data pool needed to be "swapped over" through sub-module maneuvers for as many rounds as needed all the way from influence line analysis to fatigue strength assessment. For this "rare reason" the **FEA** program **I/O** and data management features offered in the context should be much more sophisticated than what outfitted for **non-CRG** applications

As typical (**CRG** or **non-CRG**) engineering project advances in stages through preparing, reviewing onto approving phase, "Analysis" and "Design" should be regarded as two independent engineering functions as far as division of work responsibility is concerned and/or for ease of handling subordinate project reporting commitment or for management decision making purposes, etc. – even though the same (group of) individuals may take charge and/or participate in both functions.

Whereas for Proper Engineering of **Crane Runway Girders** serving Mill functions, there is a very comprehensive must-do step imbedded in between "performing" **Analysis** (**for deformation and stresses**) and "justification" of **Design** (**against metal fatigue**.)

> In that background, the adequacy of "design against metal fatigue" cannot be formally started or can never be finalized on record ahead of proper "calculation" of fluctuation/reversal subsisted in various states of stresses
>
> That is very much like getting caught up in an engineering loop-de-loop situation. But one can only be so fortunate if the **FEA** software has facilitated to "automatically" follow up with and carry out the as-said must-do steps so to avoid messing with the "extremely" tedious chores using our own version of post-processing utilities. Being fortunate? Yes, it is, all because:
>
> - Such calculation journeying from "initial analysis to final structural qualification" has a lot to do with *post-processing* a "whole lot of numerical intelligence" – *to have a feel for what it takes to do without using any data management schemes running on a genuine database development platform, just try achieving the ultimate engineering objective for once using manual means to find out*
>
> - To finish the course of an all-inclusive turnkey **CRG** design mission with minimum data management-related interruption is the main objective

> And thus in "every" *post-analysis-processing* step, one would be better off to establish and maintain proper data flow control seamlessly at all times with aid of robust database management utilities as to streaming among (1) influence line-moving load enveloping data, (2) stress combination data and (3) fatigue strength assessment data and so forth

The question is can any regular **FEA** software offer the needed amenities and achieve the goal

**FEA–driven CRG Engineering Practice** should fit in as a very serious "user-beware" or "better know what we are doing" state of affairs.

Relying solely on on-the-market software's "raw data **I/O** convenience" out of "initial analysis" is not enough. For qualifying **CRG's** structural adequacy, simply heed the warning that:

> Those software packages either having (1) scanty forethought on facilitating data-information-intelligence retrieving or having (2) severe limitation on data post-processing and customization for our unique **CRG** "design qualification" purpose should be the ones to avoid

The bottom line: *One must choose the proper tool.*

## 5.13 Finite Element Analysis – Tool of Approximation

From the very beginning up to the present, so long as automation tools were developed with a goal of delivering utmost usefulness for Engineering Applications then, on conditions, any well-developed or programmed **FEA** software should be able to (1) deliver highly accurate results and (2) work out solution with respectable turnaround in most cases.

On our recognition of the functionality and serviceability that can be provided on our behalf, **FEA** has its place firmly established as an effective tool of choice in tackling tedious structural engineering tasks.

Yet on being useful or not for users having various interests and backgrounds, employing **FEA** as a tool only makes sense provided that the given Structures or Structural Problems could be idealized suitably – including the **Boundary Conditions** – into analytical models for practical and analytical purposes.

On the down side as to full-heartedly relying on **FEA** for a typical application − **CRG** or **non-CRG** in the same context − one could easily defeat the primary purpose from:

- Not choosing the appropriate **FEA** tool(s) suiting the specific nature of problems,
- Not following specific modeling convention or observing limitation set by the tool,
- Misinterpreting the result, and
- Not able to authenticate the result through independent review, etc.

Other than all that, hereinafter as highlighted is some of the **FEA–specific** "dos and don'ts" on behalf of **Crane Runway Girders** of general interest:

Beyond fulfilling pure analytical and problem-solving obligations, most modern-day **FEA** tools were enhanced with extra non-analytical-purposed post-processing modules packing features geared toward various application intends and rationales

These tools may differ in user interface, **I/O** format convention and the Construction Material-related Design Code-dictated/solution scheme, etc., but without looking under the hood, all analytical-driven amenities came with the elected tools should adhere to the familiar Engineering Mechanics Fundamentals revolving around *force equilibrium, geometric compatibility and material elasticity*, etc.

And most importantly among all options and choices available on hand on **CRG** behalf, one should never fall for blanket proclamations from software vendors without "seeing" the proof of a couple of sample problem-solving **I/O** demonstrations executed on our terms or test-driving the tool to the ground to our own total satisfaction

Prior to committing a specific tool toward fulfilling our **CRG** obligations herein, we should always (1) avoid being sidetracked by the fancy demo too far away from our analytical realism and (2) try on all **I/O** facilities or furnishings and "see" if it could truly handle the *analysis of unsymmetrical sectioned girders subjected to moving load, torsion and provision of design against fatigue*, etc. If not then, watch out for garbage-in-garbage-out situation

Thereby among all as offered, settling on an appropriate **FEA** tool that could provide the exact features and functionalities meeting our full-scale **CRG** design/engineering objective seemed like a never-ending aspiration.

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If only we forsake the bells and whistles on building material-dictated structural design and **elaborate CAD** features or whatnot but try to understand from a pure analytical viewpoint then, **FEA** had been exploited for practical engineering purposes beyond **R&D** reasons to provide approximated solutions that more often than not were motivated under some of these common objectives:

- When closed-form solution or exact answer to the given problem on hand does not yet exist and/or is beyond reach for time being

- When validating/debugging/reviewing the results obtained independently by other means or from using other tools, etc.

Anyhow, given that the "analytical results being approximate" in nature by means of **FEA** through numerical method, and so by the significance of that phrase, using **FEA** pretty much laid down the fact that the "solution is only approximated" versus the "exact solution" through close-formed formulas.

How accurate or how approximate an engineered solution we end up with from using any "software tool" is only relative, yet highly dependent on how accurately the given problem was idealized (by us) into the specific model to begin with, after all

"**FEA** modeling task" for structural engineering applications usually consists of these major ingredients: *Geometry, boundary conditions, material and loading*

Geometry wise, a structural **FEA** model is built up from smaller elements such as stick, skeleton, mesh, solid or their combination. Material-wise, it is not the real thing but a numerical entity with attributes in close resemblance − if not the closest − to the real structural substance

In the end, "the degree of accuracy" or "the degree of approximation" as exhibited in the solution to our problem has a lot to do with the "reliability of enumerated data" beings coded by the Users per specific **FEA I/O** mandate.

## 5.14 Finite Element Analysis – I/O Flaws

To so many structures having (1) plainer attributes in geometry and load conditions with (2) much simpler *user-assigned* problem definition, the setting up of analytical model for which can be rather straightforward in terms of the demanded efforts, required number of takes to prepare, review, revise, resolve and finalize the input data.

Whereas in dire contrast as in the development of much larger-scaled **FEA** model with unusual characteristics – such as geometry, load nature and design mandate, etc. − for which the compilation of input statistics might reveal a very different story.

> The demanded efforts in **I/O** data assembling process to accommodate the given conditions could be very much involved; but at any rate, although easier said, it can still be a controllable exercise if the task can be modularized into manageable steps whichever as seem fit specifically for **CRG** applications; and that may involve:
>
> - Sketching out key regions or zones where weldments and bolts are to be located/installed
> - Identifying the boundary of effective section based on compact element criteria
> - Conceptualizing how the structure is to be idealized including the **boundary conditions**
> - Streamlining the modeling concept into discretized element or meshed network
> - Digitizing the pertinent geometric features, loads together with material attributes
> - Consolidating a final data input packet
> - Transcribing packet data into "texts and/or numerals" for software retrieval, digestion, processing, post-processing, storage, etc.

During the planning stage and the pursuing of a "pseudo perfect" **FEA** model on **CRG's** behalf, many of us – experienced and inexperienced alike – often toil between two extreme modes of emotion long before the "input data organization" was finalized or completed; i.e. feeling exceedingly fortunate if all went well indeed or relentlessly nervous wondering if any errors were (actually) committed or not.

On preparing **FEA** model:

> Provided we were able to compile the input packet flawlessly, otherwise any nitty-gritty **I/O** related hitches from "our" initial carelessness once being carried along with/into the subsequent steps/moves – whether through automated utilities, graphical or manual means – would for certain bring on unwanted outcome
>
> > The biggest issue with that is, as long as the input syntax rule was correctly followed, the software might still accept the erred set of input as is and may run it with no complain, so imagine can we trust the result
>
> Making **FEA** model is like finishing the first leg of a long-drawn-out **CRG Engineering** relay contest. To any self-induced oversight entrapped in the nerve-racking passage, unless we could track it down in time, or else they simply add up to the tolls due or not yet due; some could lead in a chain of numerical flaws that can bury themselves not so conspicuously in turn hampering the general progress

Critiquing from a different angle:

> As always, the accuracy of results obtained from numerical-based **FEA** could never rival that attained from closed-form solution albeit the **FEA I/O** had been prepared "flawlessly" free from "alleged" human errors
>
> Thereby if only we could minimize the possibility/probability of committing human errors – whether numerical, logical or conceptual − in model-making through extra vigilance; but even so, we still have no effective control of those seemed never happened numerical rounding off and truncation errors inherent in the *equation formulation, matrix solver, software programming language and hardware limitations, or any of the mishaps of non-human source*, and so forth, which can be triggered by our "alleged" human errors

Somehow and anyhow we would like to make the **FEA** work out to our total satisfaction in the end despite those probable snares and imperfections that may bestow. And if not being too picky then, the final "work of art" on our structural idealization task should deserve a passing grade for being good enough or technically adequate provided that:

(a) The "analytical model" – including the boundary conditions – as assembled was free from logical and/or artificial geometric modeling data irregularity

(b) The "collaborated input information" as compiled for analysis was free from misinterpretation of the pertinent design intent

On the other hand,

> Every so often it takes barely a "minor slip" to revert the as if all good-to-go state into a seriously wrong situation from what seemed perfectly all right. The worst is when **FEA** software takes it all in and usually does not care if we erred in our data enumerating process with the exception when there is some very obvious syntax error
>
> On balance on account of certain inevitable "minor slips" – as it passed through the input scanning phase so long as it didn't "violate" the **I/O** syntax construct – although **FEA** might still be able to run the process as usual and turn in the so-called "approximated solution/result" from which after all, but for the worse part, this sort of "approximation" would bear very little or no resemblance to the anticipated "structural reality"

Other than those simple and explicable numerical errors on *geometric measurement, material parameter assignment or loading calculation*, etc., quite often the alleged "minor slips" may be more acute to the analytical results and the final design – due to serious conceptual flaws and/or critical modeling defects – for example:

- Wrong element type(s) were selected for the problem on hand: Adopted thin plate or thin shell elements instead of exploiting solid elements of higher order or those with more **proper** shape function setup

- *Questionable application of node-element connectivity logic thereby inducing "numerical difficulties" or causing considerable "stiffness disparity" between adjacent/neighboring elements affecting numerical accuracy*

- *Unrealistic or wrongful assignment of boundary conditions* resulting in some cases unreasonably large displacements led to misleading judgment or conclusion

- Other unwary but fatal logical faults causing general numerical instability or the solution failed to converge

Consequently, blindly compelling an engineering/design outcome based on unwarranted application of **FEA** shouldn't be mistaken as something being the ultimate end-all, be-all or cure-all solution means.

How reliable and how appropriate or how unreliable or inappropriate any **FEA** tool is for solving a specific type of structural problem, such as **unsymmetrical sectioned CRG subjected to torsion** in particular, relies heavily on at least a few more palpable specifics:

- The unique post-processing feature of the facilitating software and
- The unique problem on hand and/or the limitation of software analytical facilities

Those are fairly standard caveats that should apply to **FEA** coded of all purposes on all structure types. But to see how that has any further stimulus upon our **CRG** interest and to sketch out the broader specifics on our own behalf, we need to fall back real hard on what we should be most familiar with:

- The processing logic from what we normally do in the design of other (**non**-**CRG**) structures
- Torsion behavior of open sections
- Qualification of structural components against metal fatigue

## 5.15 Finite Element Analysis – Latent Difficulties

Given an "ideal" situation off to a good start, by which our **FEA input data** had been "meticulously" coded free of modeling flaws in every aspect considering geometry, boundary conditions, material and loads, etc. Just so be it a talking point.

> In automated engineering process, provision of bug-free **FEA input data** is only the beginning of an aspiring attempt towards a commendable ending only from an analytical position, which is yet to be unveiled not until the long-drawn-out engineering-design process is all over with

Once the groundwork is laid "perfectly" through bug-free **FEA input**, barring the mystery for it is still far from reaching the closing show-and-tell phase; and thus a good springboard to make an preliminary-intermediate assessment is at the completion of initial load response-stress analysis.

Intermediate assessment is an essential course of action when we can catch garbage-in situations just in case there's any **bugs**; for one thing what concerns the most should be; *what kind of final design outcome that the **FEA input** may lead to*, would it be a garbage-out?

> Unlike solving homework problems, real-life engineering result cannot be half-right-half-wrong or a-lot-of-right-and-a-little-bit-of-wrong and still call it good. Regardless to how the input was assembled at the beginning, we only "see" that the end results could either be perfectly all correct or be totally incorrect
>
> What controls the final outcome depends on a number of measures transpired throughout the development, and more so on how had we – or had the software – dealt with the handling of transitional data following the initial analysis on way to qualifying our **CRGs**
>
> *The biggest issue is when too many among us don't "see" clearly what is being totally incorrect*

Inevitably user-introduced **bugs** or **mistakes** do come about even in a carefully planned engineering process via **FEA** means. These **bugs** or **mistakes** can be broadly categorized into either the *logical* kind or *enumerating* kind.

> Some of the faults are "placed in" through incorrectly prepared **FEA input**, misuse of **FEA tools** and/or mishandling the intermediate results, etc., or else "flaws" could still develop in the post-data processing stage. Impending troubles would take its toll should we not catch and fix them in time, even though unaware of their presence at earlier stage

The main interest herein is by applying "commonsense" only then to "discover" how "easy" is it to fail the qualification of the design of a **CRG** "the hard way" when using **FEA** tools.

There is no hurry to sink in too deep too soon on the issue; what needed is taking it in steps and developing a good sense in identifying which brand of difficulties that we may face when working with **FEA**; and then "see" if there is practical way out, or for the worst <u>if not at all</u>:

(a) <u>Starting from the most basic chore:</u>

Be familiar with how to carry out <u>simple calculation</u> is extremely important; for example, of the fiber stress due to (1) flexure bending moments using formula ($\mathbf{f_b = M\ c / I}$) and likewise (2) the warping normal stress due to bi-moments from ($\boldsymbol{\sigma_n = M_{zw}\ \omega_n / C_w}$) are the so-called <u>simple calculation</u>. Once adapted to the routine, it should be relatively trouble-free to repeat the same chore over again for as many times as required even when carrying it out manually.

When in a state of hesitant albeit all went well, still, many of us may suspect if, for example, there might be a connection design issue, not necessarily an actual error, or else being driven with a habit of double-checking things all on our own urge, even if there isn't any error surfaced yet.

One of the stumbling issues that "bother" many of us is whenever the unavoidable torsion enters into the picture; it could nurse up uncomfortable speculation if any oversight subsisted in the **FEA** results. So how do we know is there a **localized** problem in the first place? If so then ask ourselves some **logical** questions such as;

*Was that caused <u>actively</u> by the input from us using/interpreting the tool incorrectly?*
*Or that caused <u>passively</u> from limitation of the software that itself is a wrong tool to begin with?*

(b) To extrapolate from those two questions listed above, suppose we had proceeded with all as correctly as planned, much as we think we did that; but, how do we know if all is well indeed following a typical **FEA** run?

Armed with our general knowledge in the treatment of **CRG**, together with our attention span and patience combined could play a major role affecting the efficacy in discerning what is right and what is not right following a **FEA** run. Depending on our specific interests while assessing the structure's adequacy through each stage during normal debugging process, we could be looking for <u>alleged</u> bugs in several different ways by:

- Either combing though the detailed text reports obtained directly from software vendor-supplied standard **I/O** stream or those made-to-order text results/reports based on users' on-the-fly querying prompts if there's an option available for such intent

- Or trusting the innate Graphical User Interface (**GUI**) facilitated by the software tool or added-in specialty graphics tools by third party

- Or going deeper into post-processing the **FEA** results on our own by downloading and (re)organizing of which into useful "dataset" suiting specific levels of details **vastly CRG-specific** (*torsion related or fatigue related*) – a very costly undertaking

  One may start retrieving a subset of text-based enumerated data if already laid out in inherent formats based on specific sorting sequences, then we can consolidate and rearrange the crude data using formats/styles that goes well with our personal preference into "suitable intelligence" that deemed most effective and most convenient for a full-blown evaluation session − all done via special "Utilities" either canned in the software tools or that customized by others (or on our own)

To have a greater success in dealing with **CRG** matters in general, it won't take long to "learn" that the highest priority in it is to take in custom-made robust database management facilities before structural evaluation chores actually begins

<u>In order to customize well on our own dime</u>

The progression may involve (1) tidying up in proper order those underdone stress components, (2) conditioning data hierarchy per tensile fluctuation/shear reversal grouping logic, and then (3) readying up for the conclusive fatigue strength assessment.

Whereby (1) data scanning and (2) data record (re)organizing (however it's being done) turn out to be our "immediate projects" subsequent to a raw **FEA** run.

To prime up an impartial ruling on what yet to be unveiled in wake of a raw **FEA** run, everything that has anything to do with within our engineering scope would have come down to the texts in view and/or the graphics perceived:

Standard text report − whether of force-based, stress-based or deformation-based − does have its merit as being a part of the electronic (or paper) trails

But while browsing a lengthy text report, it is way too easily getting lost or into a plain boredom by too many (screen) pages of monotonous digits and decimals, especially if the contents were not sorted in sequence aptly per our spontaneous querying benchmarks, which only make better sense when the criteria in use is most agreeable with our instantaneous "what if" rationales – usually that can only be drawn on the fly based on our spontaneous probing criteria

Been there, agreed? Then even if the spontaneous query logics were put perfectly under our total control, but, it always takes time to zero in (or opt out of) a potential problem area. It gets much harder if the vital info were buried deeply somewhere in one of those generically-formatted documents (pages or modules,) especially when the organization of which were not down-to-earth enough for general **CRG** engineering-design evaluation purposes

Then instead of browsing the text reports, a quick zooming in and/or panning out of the deformed shapes or hot-spot stress plots could be quite helpful in channeling our focus into areas of general concern or wherever that needed our immediate attention, but only if we don't get lost from doing it in the first place

Most of the graphics and plots were in no doubt eye catching, but still far from telling the full story we "wanted" on behalf of **CRG**, such as indication of where is the fatigue hot spot? We meant fatigue, not the non-fatigue

Graphics or no graphics, what worked well in vivid colors as mental prompts or visual stimulus at best could never be sufficient enough in providing integral solutions or conclusive diagnostics that would serve the total **CRG** interest conjured from *moving loads, serviceability,* ***reversible stress ranges and fatigue***, etc. unless the software package has been geared/tooled up or tuned up for those tasks. But then which software that does it on **CRG** behalf? And if it does then:

Can we pinpoint at a chosen nodal point and "see" what's going on there?

For example, could the **GUI** actually "tell" us what is the maximum tensile stress fluctuation value evaluated at a specific bolt hole or weldment near the tip of bottom flange located at 2 feet from the support end due to load combination number 2? …

If so then could the software flag off a warning icon that the tensile stress fluctuation does or does not exceed the fatigue allowable stress range after experiencing 500,000 live load on-off cycles …?

So as not to place a rapid-fired criticism on the shortcomings of those dull-formatted text reports or fancy on-screen graphics, but unless the "chosen software" has been **customized** to our exact **CRG** engineering needs, the bottom line cannot be drawn yet since still we are more interested in:

- Retrieving **correctly** and **orderly** the extreme (maximum and minimum) values of all different categories of (**X**/**Y**-shear and **Z**-fiber) stress matching all individual load sources and combination decrees (flexure and torsion, etc.) and

- Following the natural data flow path and node-element modeling logics and their data relationships from among the various categories of stresses and displacement statistics, etc., on and on, all and more for ultimately prepping the groundwork prior to (final) structural evaluation progresses

Once "completed" a successful **FEA** run, if we could

(1) Fully count on the software tool's (limited) searching, querying and/or sorting facilities and (2) pin down the precise locality and the intensity of all categories of stresses at selected node or element of our interest without getting off course

Then what a success we are in already.

But that is not all as-wished-for yet; imagine how nice if we could (1) make sense promptly from the mound of **FEA** output without post-processing the "raw" data on our own dime and (2) able to figure out the straight stress sum for non-fatigue intent and the stress reversal range for fatigue evaluations, etc., would that be viable, if (not) impossible?

*Finally, to attempt consolidating the intermediate results following **FEA**, if lacking an organized data **I/O** facilities setup properly matching (exactly) our/your **CRG** design qualification process flow schema then, it would only be wasting time and resource.*

(c) In a real-life (non-textbook oriented) project, the toughest part of using **FEA** is the tremendous effort needed when the original geometry configuration is to be **optimized**, such as making change in element thickness, length or depth, etc. *So do we need multiple models?*

For all real-life model updating events, not only multiple versions of model geometry input data needs to be kept track of, but also the analytical results corresponding with each model needs to be stored individually and correlated through proper database links for cross referencing and decision-making purposes – otherwise watch out for a potential of running into some garbage-in-garbage-out situation.

## 5.16 Finite Element Analysis – Garbage-in-garbage-out

Almost always – and only if feasible – whenever tackling labor-intensive or time-consuming engineering problems, more and more engineers would intuitively opt for **FEA** for a fast track solution; thus to much greater extent, many Practitioners believe that **FEA** must be a **perfect** fit for real-life **CRG** applications also, sounds reasonable.

*And then knowing the fact that (1) **FEA** does not produce exact answers compared to what closed-form solution does, and as already proven that (2) CRG in so many ways is more exceptional than most other ranks of structure*; although to agree or disagree with the former opinion is only an individual choice, but in all fairness, it would take some interesting debate and clarification before one can fully endorse or denounce the selected platform as the **perfect** tool for **CRG**

Anyhow, **FEA** may or may not be the winning choice contingents on a number of variables. Prior to adopting the **FEA** platform over other means, one should at least confirm in the first place that the showcased **FEA** tool of choice was "indeed" appropriate for **CRG's** (1) analytical and (2) engineering-design assessment purposes.

Before the "Enter" key is pressed – think twice – we "need" to convince ourselves and the peer reviewers that (1) the **FEA tool's custom-mandated** input geometry-loading information had been properly prepared suiting specific purpose to start, and then making sure (2) *the given **CRG** entity would be qualified meeting not only the allowable non-fatigue and fatigue stress criteria but also the serviceability constraint*

Unless it is a **turnkey tool** that calculates tensile stress fluctuation and shear stress reversal for fatigue strength evaluation purpose, otherwise it is not final; we "need" to make sure all the not-yet-resolved intermediate results developed progressively throughout all groundwork stages of the automation process were "reliable enough" to be advanced into the final stage of engineering

No matter how and what we "think" we had perfected the basic **FEA-custom-mandated I/O** chores aspiring towards all rightness as presumed, albeit however much effort spent, the "*bottom line burden*" is still on us to prove what we had accomplished does turn out for the best benefit of **CRG.**

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**First and foremost, assuming an <u>appropriate</u> FEA tool was already chosen (or in use) for the task, and let's see.**

The immediate course of action following a **FEA** run should be to make certain we were not in a garbage-in-garbage-out situation − the **CRG** kind − <u>especially in the treatment of torsion</u>.

What in need is a hands-on strategy to check things out; *knowing that the fix for <u>unsymmetrical sections</u> is ultimately we are most interested in, but, before doing anything with it, let's see what is in on with <u>symmetrical sections</u> first* – remember there are two kinds, **singly symmetric and doubly symmetric**.

> When finished with the initial analysis involving *moving loads* and the subsequent *fatigue-conscious* load combination process;
>
> > *Again, **unless** the evaluation is for <u>symmetrical sectioned</u> structures subjected to torsion-free **simple bending*** – just remember the golden rule, the load resultant must pass through **Shear Center**
>
> Otherwise whenever torsion is playing a part then,
>
> > One would not be lucky enough in knowing (or guessing) immediately (1) where the straight-up <u>maximum or minimum</u> stresses are located (imagine *out of so many stress categories be they fiber stresses or shear stresses*) and (2) let alone knowing where these extreme numerical incidences were posted, whether it occurred at or near which node(s) within which segment(s) or zone(s) among all **X/Y/Z** spread, there would be a big unknown
>
> In other words for **symmetrical sectioned members** subjected to **simple bending**, every structural engineer should be able to cut it briskly as far as the locations of maximum or minimum stresses are concerned;
>
> > That means it would not be an issue as to **verifying whether if we were in a garbage-in-garbage-out situation** even with no prior knowledge of their **X/Y/Z** whereabouts − wherever that may be located between **Z** = 0 and **Z** = **L** – and it work fine <u>provided that all loads pass through **shear center**</u>
>
> As matter of fact, we can pick out an arbitrary **z**-cross section slice to get the game going; and one should have little problem identifying where **exactly some** (or all) of these internal stresses that should have <u>diminished to zero</u>, at least **theoretically**, got the clue? This initiative should work out well for **symmetrical sectioned members** subjected to **simple bending**

*Assuming there is no global buckling or local buckling/post-buckling issue to deal with*, in theory, again − as we recap from the very fundamental structural behaviors of typical **symmetrical sectioned members** and extrapolate the familiar cognition into **unsymmetrical sections**:

(a) While focusing on simple flexural behavior free of torsion's hindrance, the elastic bending stress would diminish to zero at a cross section's **elastic centroid**;

Over there, zero bending stress implies zero bending strain and thus no change in fiber length, i.e. there is neither elongation nor contraction in longitudinal fiber at the **elastic centroid**, but, **globally** all nodes are to displace and settle at wherever the "final" equilibrium may take them to.

In short: The condition of zero stress/strain from **flexure bending** effect must be confirmed **internally** at series of **elastic centroids** that stringed along the full member span from end to end, otherwise something is wrong; or is that a hint of **garbage-in-garbage-out** then?

As straightforward as already understood but, we should make sure (1) to correctly locate the **elastic centroid** and (2) as applicable to exclude (or include) accordingly the influence from axial ± $(\mathbf{P_z} / \mathbf{A})$ stress during the evaluation.

(b) Next onto torsion, for which **Shear Center** (**SC**) is always the center of attention. For open-sectioned members – *to simplify things, assume there is no influence from flexure* − **internally** throughout the member length there are two conditions of interest to fall back on:

- There is no rigid-body **Z**-movement **locally** at the **SC** and
- Warping normal stress must = zero at **SC** implying zero longitudinal strain

Easier said and also easier understood for the two conditions are always valid considering normal structural behavior under torsion, should be no question asked there, at least for **doubly-symmetrical sectioned members**.

Something seemed suspicious not being said out loud here, these conditions would be invalid when applying **flexure analogy method** to cross section profiles other than doubly symmetrical geometry, which for good reason if taking that as a homework problem; all Readers should be able to tell why.

The key is, **shear center** must be suitably located in advance and the warping normal stress must be calculated using the correct value of section properties and the applied torque, too.

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In a more generalized setting such as **unsymmetrical sectioned CRG** supporting moving loads off **shear center**, would that be a problem?

> To accommodate what needed in the "total" design qualification, the setup of data relationship would be so much more sophisticated in order to "handily" deal with the intelligence connecting *applied loads, load combinations, boundary conditions, enveloped structural responses and various brands of internal stresses and the notorious stress fluctuation/reversal situations*, etc.

Evidently from that vantage point, we should anticipate that everything could appear as if so much obscured by everything else whenever intermingling flexural with torsion. Knowing ahead of time which category of stress that must be reduced to zero at specific coordinates as just pointed out earlier would become impracticable, all because of this modeling issue:

> *It has a very slim chance – or else if only by chance – for the innate coordinates of either **elastic centroid (EC)** or **shear center (SC)** to match any input domain coordinates built based on user-defined nodal/model geometry*

Nonetheless that is not a good excuse to recoil from what needed to be done; and once the **CRG FEA** analysis has come to a stopping point, the "first-thing-first accountability check" would still fall on us to verify the accuracy of analytical results, a very tedious chore indeed but not impossible, and so be it.

> Despite the incoherent barrier of **EC-SC** geometric offset standing in the way, that "first-thing-first accountability" could still be fixed up for linear stick member through checking (1) the external force equilibrium and/or (2) the approximate or close-enough value of fiber stress depending on the level of details enticed

For matter of interest, we could always tell from the free-body diagram if there is an equilibrium issue simply by comparing the applied load sum with the total reaction added up at all supports – except that, it is not so straightforward for a meshed/**3D** model. Then from a flexural stress standpoint, one could always do a quick run of (**M c / I**) calculation for a linear stick and simply check it against the output, but once again, it is not so easy for torsion especially when applying the similar trick to a meshed/**3D** model.

The morale: Nothing is ever easy only if we do the real thing the real way.

There just doesn't seem a universal magic wand exist for validating analytical results due to torsion, even for a linear stick member in some cases. And if **FEA** is carried out for unsymmetrical sectioned **CRG** structures then, not only the magic wand for checking torsional response fails to exist but also flexural response's confirmation would probably be in jeopardy, too – for reasons to be explained − therefore the only reasonable means to get it done is through all the "Zeroes" in terms of structural deformation, correct?

As we evaluate structural deformations whether of translational or rotational **DOF**, each and every "Zero" entity is evaluated at nodes confined as coded in in the local **XY** plane. What needed be identified is, some of these zeroes must apply at the **Centroid** (where **c** = 0 in **M c / I**) and the rest of zeroes must concurrently or exclusively apply at the **Shear Center** (where $\boldsymbol{\omega_n}$ = 0 in $\mathbf{M_{zw}}$ $\boldsymbol{\omega_n}$ / $\mathbf{C_w}$) but, again, this criteria works only for symmetrical sectioned members, again, why?

Bear in mind for unsymmetrical sections as already mentioned,

The coordinates of both ***elastic centroid*** *and* ***shear center*** *had no chance or a very slim chance to match any input domain coordinates built based on user-defined nodal model geometry* thus (1) there is no practical way to locate where "exactly" c = 0 and $\boldsymbol{\omega_n}$ = 0 and (2) there won't be any **FEA** output available for (fictitious) non-user-defined nodes – off topic, unless we do it the hard way using interpolation

Seriously then, do we have a problem with verifying analytical results for unsymmetrical sections?

In conceptual sense, it is not difficult to tally up and recount the requirements or strategies in identifying "net zero deformations" but,

How to transform the conceptual "zero-value-requirement" per static equilibrium into real-life practice based on (1) the collection of as-built **boundary conditions** and (2) the universal modeling scheme applicable to any specific **FEA** model may not be that straightforward. See the long dark shadow casted behind?

Ideally, the suite of **boundary conditions** – assuming it is properly specified − needed be set up (technically) witty and tricky enough so the peer reviewers can make up quick ruling based on the expected structural reactions/behaviors at the girder support ends, and see clearly what established in the model is correct or if not so they know what to look for

Finally, again, do we have a problem with verifying analytical results for unsymmetrical sections?

Yes, that's because in most cases, *hardly the* ***shear center*** is lining up with ***elastic centroid*** along one of the principal axes for a fact

As understood once more,

On comparing the results given from **FEA** run and that obtained from using closed form solution, there is no denying that **FEA** does provide results in approximation

How approximate or how realistic the **FEA** results are and will it meet our total satisfaction or not depends more specifically on (1) how competent is the **tool** so as it fits for the application purpose and (2) how realistic the analytical model can be constructed per user input intent – *especially the* *boundary conditions* – and how realistic is the output result as compared to the real behavior of structure under intended load in real life. So isn't this a mission impossibled?

Now let's delve into some of the finer points on the **tool** issue:

- First of all:

  Things change and evolve for the better; while not knowing if any (newer) tools/packages in the open market were better or worse than what have we in the past, and unless there is one **FEA** "tool" already chosen (*appropriately*?) on **CRG** purpose otherwise one would be extremely overwhelmed in the searching from scratch, given so many commercially available software packages to evaluate and choose from

  Among all that are out there with so many (name brands or ones unheard of) choices, searching out for the one that offers preferred **I/O** formats, utility options, design features and most importantly the choices and **limitations** (shortcomings) of *element shape functions* and element/nodal configurations or arrangements, etc. becomes an adventure in itself

- Secondly:

  Take into account the verity in evaluating software packages, the fact is, each candidate tool has its unique features, **I/O** conventions and inherent limitations, etc.

  No matter how ideal the model geometry and boundary conditions were specified to best represent the structural configuration and the support condition together with loading-load combination input, etc., in spite of everything being done correctly from users' perspective, but deep down under, there is always a lag in accuracy between the **FEA** results and the **true structural behaviors**. So seriously, only if we know what the **true structural behaviors** is, otherwise this could easily lead to a blind leading the blind kind of adventure

- Thirdly:

  Element Shape Function is the soul of **FEA**

  As "tool" users, normally there is no need to open the hood and dive into convoluted subjects such as matrix solver or construction of stiffness matrix, etc. only we can appreciate the fact that matrix constituents and the accuracy of analytical results were rooted deeply in the shape function of the specific type of finite element(s) chosen

  Shape function in a nutshell is an "element-unique displacement function" articulated usually through polynomial expression. The element-unique function is the means employed to interpolate between or among the nodes where their displacements were known or already been solved as result of (stiffness) analysis. A generic polynomial function could be of linear, quadratic, cubic or higher ordered form(s)

  The development logic in each unique shape function varies from form to form, depending on which polynomial terms (1) were retained or engaged to match the complexity (or simplicity) of the element's geometric configuration and (2) were fine-tuned with special coefficients or constants to emulate the element's structural behavior through deformation pattern; therefore it is almost like what we choose – correctly or wrongly − is exactly what we will get

Once again as "tool" users, there is no need for us to peek into what's under the hood, but it's worth knowing that what would be most confusing to the Engineers is that by the innocent appearances, a number of look-alike element(s) could be mistaken into some other element(s) of similar contour and shape but were developed out of drastic different shape function(s) formulated in consideration of a totally different behavior (under flexure and/or torsion)

Now here comes a semi good or semi bad news;

Unless there is access to some of the proprietary **R&D** releases, not every **FEA** software package from the myriad of that could evaluate torsion behavior correctly for unsymmetrical sectioned member especially in area of warping behavior. Missing the warping rigidity in **CRG** applications may (or may not) make the result more conservative in certain aspect for certain part of symmetrical sectioned members, but it sure does not earn the ticket to the moon yet, why?

Because (1) it could miscalculate the lateral/vertical deflection at the top of the rail and very likely that could exceed the limit per serviceability requirement and (2) whether considering warping's influence or not, there is not yet (as of this writing) officially documented **FEA** test runs and evaluation (justification) on unsymmetrical sectioned **CRG** under torsion influence without engaging the soul of torsion, **shear center**

But despite taking in all positive and negative senses of arguments, if we insist on certain software package to be the chosen tool that can do it all then it may be necessary to consider and confirm some of the followings as minimum:

- The tool should offer choices of a number of higher-ordered solid element as modeling bases and the chosen element type should tender sufficient number of nodes or number of degrees of freedom compatible at the interface with other lower-ordered elements

- Stick with (isoparametric) element types to minimize confusion in that the number of nodes for geometric input matches the number of nodes where displacements are to be solved for

- The shape functions setup for selected element should be sophisticated enough to take in both *warping and flexural behaviors* duly created with proper polynomial

- Most importantly the software should offer options and instructions in how to properly specify the boundary conditions both in general application and in special cases, e.g. *what happens if the coordinates of **shear center** are known, and what if not known*

Supposing the **FEA** tool selection had been squared away, approved and ready for setting up the model geometry, mesh connectivity, boundary conditions and the (moving) load schemes, etc., what needed the most after that is setting straight with respect to our "input coding strategy" that would simplify the task of justifying and validating the analytical results after each **FEA** run.

For symmetrical sectioned members, again:

When coding in terms of physical nodal geometry, the "strategy" must take in deliberation of how to effectively embed **X/Y/Z** coordinates where $\mathbf{c} = 0$ and $\boldsymbol{\omega}_{\mathbf{n}} = 0$, all about reflecting those aforementioned "Zeroes" (went back a few pages.)

After all, these "Zeroes" are our defense against the garbage-in-garbage-out situation that we touched on at the beginning – and yet, it's easier said than done without knowing where **elastic centroid** and **shear center** are located beforehand; *see the problem?*

To accommodate input coordinates where $\mathbf{c} = 0$ and $\boldsymbol{\omega}_{\mathbf{n}} = 0$, we must strategize a selective group of nodes took after special numbering patterns out of all nodes dispersing all over the **3D** space

Evidently, we need to make certain that at every applicable **XY**-slice of girder section, for which a minimum number of "special" nodal points should be provided with **X/Y/Z** coordinates appropriately matching the applicable section's **centroid**, **principal axes** and **shear center** where flexural fiber stresses and warping normal stresses are supposed to be "zero," respectively, *If not so then, how else could our modeling approach be validated against?*

In order to complete the chore fittingly, does that mean we have to locate ahead of time the **centroid**, **principal axes** and the **shear center** prior to completing or even starting the model construction? Then to serve all **FEA** purposes, shall we do the same generically even for unsymmetrical sectioned members? Well, yes, certainly, but there is a big problem – Readers are to dwell on this

See the "*hidden **FEA** dilemmas*" with unsymmetrical sections, yet?

When an open-sectioned stick-like member is under both flexure and torsion, and regardless to what **Boundary Conditions** were specified in any **FEA** model, we all "knew" by now that for unsymmetrical sections the **centroid** and **Shear Center** are always at offset to each other at all cross section slices; so on this logic,

How can fiber stress be "zero" at **both nuclei** at the same section slice in the same run?

Thereby it looks like if this approach works for flexure bending stress then it won't work for warping normal stress or the other way around, but is that true, or false, or totally wrong? (As matter of fact, there is nothing wrong in the thinking. Readers are encouraged to find out why not.) But shall we do something differently then?

One of the more rational ways to "right" the seemingly "wrong" situation the hard way is; it would be to analyze the given structure through multiple independent models:

Again, on symmetrical section's behalf, using one model with a set of flexural **BC** for flexural behavior and another model with torsional **BC** incorporating proper **SC** restraints for torsional behavior could be the answer

But, to truly isolate or normalized the issue of **BC** for true **3D** load-response applications, only logically we might need three models for a generalized girder having no axis of symmetry: One model for principal **X**-bending, one for principal **Y**-bending and another one for torsion, but isn't that practical, or impractical?

Practical or not is not the main focus in this discussion, yet from the authenticity of using multiple models alone, the "*hidden **FEA** troubles*" seemed resolved, but only on the surface. In fact we are not totally out of the woods at all.

A few more "obvious troubles" now just popped up, all of which have something to do with (1) how many normalized models are actually needed and (2) how to properly handle the analytical aftermath for a generalized **CRG** application, etc., say, in the worst-case scenario:

- *In* satisfying *all "Zero" fiber stress requirements, what would the boundary conditions be for the type of loads such as rail misalignment that could cause both unsymmetrical bending and torsion responses?*

- Remember *the post-local-buckling and effective sections, how to prepare the boundary conditions for that and evaluate, compare and combine the analytical results from a gross section and an effective section?*

| *Would this be one of the most important reasons that **FEA** becomes totally useless to **CRG**?* |
|---|

- *After the model normalization process, how to combine the effect for fatigue and non-fatigue qualification after a typical **FEA** session involving more than one model?*

It seemed like such a drag already if there is not enough warning from all of these questions and lack of more convincing answers then, shall we think twice. Seriously, not only these questions need some valid answers for the worst-case scenario but seems like some of that also apply to the normal-case scenarios, too, aren't they?

Whatever we do with **CRG** using whichever **FEA** tools should be our (or your) own choice, but the mess remaining is how to prove and justify that we are not in a **garbage-in-garbage-out situation** and most of all, how to convince our peer reviewers.

> *The "presumption" is, there is no easy way or simply no way to verify **garbage-in-garbage-out situation** for unsymmetrical sectioned girder being analyzed using **FEA***

*Not in fully understanding of and not in total agreement with the coverage on the subject up to this point are two different matters in question; but to those joining either league, it would be too hard-pressed not seeing that* **garbage-in-garbage-out situation** *is always there steps ahead; Readers are encouraged to "see" the reason why.*

## 5.17 Stiffness – As Understood

The simple term "stiffness" could be recognized "conditionally" as **normalized force-displacement ratio**.

> Once stripped into a non-dimensional entity, the leftover is barely a superficial scalar; and it would not make any engineering sense, unless paired up the scalar with proper dimensional unit into a full-scale vector in association with a proper **Degree Of Freedom (DOF)**

"Stiffness" in blurry engineering dialect is sometimes compared to an eminent phrase "spring constant" at face value, but that is definitely not a proper **CRG-purposed** equivalence.

> However we cut it, the same word "stiffness" when brought up as a detached idiom might cause confusion if it were mixed up with another familiar term "rigidity" that relates to the bundling of material property and local cross section property such as (**E I**,) (**E A**,) (**G J**) or (**E** $\mathbf{C_w}$,) etc.; so just to be clear

On **Structural Engineering** purposes, the term stiffness could be inferred into a wordy expression such as *the "amount of force" required to induce a "unit displacement" along a specific **DOF** located at a specific point of the structure*.

The tactics being used in defining stiffness of a specific structure for a specific application may differ broadly for different intents, but mainly depend on:

- The level of structural details were given
- The measure of "precision" being sought after
- The branch of Engineering Mechanics on which the application is focused in and
- How realistic the assumption of structural behavior was crafted in the calculation and how was that being applied in practice, etc.

In order to make specific **Structural Engineering** implication, one must associate the stiffness of a structural member with at least:

(a) The pin-point **X/Y/Z Location Of Interest** (**LOI**) where stiffness is being evaluated
(b) The **DOF** axis (orientation) of load application acting along (collinear) or pivoting about
(c) The load magnitude being applied along/about the applicable **DOF**
(d) The induced (translation or rotational) displacement along/about the matching **DOF** axis

Among the vital constituents identified in the list, **LOI** deserves to be the most significant of all. It must be identified by the exact **X/Y/Z** coordinates prior to making calculation.

> In the simplest **LOI** specification for a regular doubly symmetric **I**-sectioned member, the linear transversal stiffness is mostly applied through the **elastic centroid** located at the mid-span − *unless noted otherwise* – but that connotation can become deceptive and easily misunderstood for applications involving unsymmetrical sectioned member
>
> > For example, we might get by conveniently from fixing the **LOI** offhandedly with **Z** = 0.5**L** as if not to worry about the **X** and **Y**;
> >
> > And yet, this is valid provided that we intend to (1) locate the load vector at the mid-span passing through the cross section's **Shear Center** (**SC**) and (2) that also matches the coordinate of the **elastic centroid** (**EC**) there the relative $\mathbf{X_{offset}} = 0$ and $\mathbf{Y_{offset}} = 0$, otherwise, here we go again, watch out for what the devilish geometric offset between **EC** and **SC** would do
>
> In a more generalized non-simple-bending-based application, even for symmetrical sectioned members, the exact whereabouts of **LOI** does make considerable difference in results based off different **LOIs**
>
> On thinking outside of the box, we could ask ourselves some of the not-so-tricky questions and try our hands at answering them:
>
> > How different would that be (1) if the stiffness was evaluated for **LOI** at the one-third point of the girder, its mid-span or at the support ends; then (2) how about the stiffness at **LOI** located right at one of the corners of its top flange or at the center of the web and (3) for load pointing along X direction or pivoting about Y axis, etc. or whatever the Location Of Interest is? See how confusing could it be?
> >
> > Besides that, **Boundary Conditions** (**BC**) of the member and any other incidental framing "happenings" at/near the **LOI** or not could also play important roles that should not be overlooked
>
> From being ever so spoiled in dealing invariably and exclusively with flexure-related issues for too long,
>
> > We might not raise question about it, whenever we made reference to the stiffness of a stick-like member, we automatically "assume" the load is passing through the **elastic centroid** and nowhere else; besides, we might also take in a default position that the "loads" were "assumed" to pass through the **shear center** in preceding instances
>
> But what if the loading situation is not so amiable to **shear center** then, would that become much more complicated?

Accordingly, barely mentioning the catch phrase "stiffness of a member" without identifying (1) the **LOI**'s coordinate, (2) the **BC**, and (3) especially the **load-to-shear center relationship** would only be exceedingly misleading; as matter of fact it would make very little sense and can lead to a totally wrong design conclusion. Once all other attributes were identified, and lastly, the dominant load vector's **DOF** should be established as:

*(a) Either "linearly in parallel with" an axis of chosen*
*(b) Or "rotating about" an axis of chosen*
*(c) Or more importantly the rare encountering of P-δ situation*

When allowing for diverse load-deflection relationships suiting various applications, the resulting **force-displacement ratio** may carry dissimilar pairing of dimensional units such as “lbs/in” or “kip/in” for linear spring or “lbs/radians” for rotational spring, etc.

Thereby in the presentation of calculated result, one must be very careful in choosing which dimensional unit is to be used, and making sure where the decimal point and/or significant digits are to be left off or left out as seem fit for practical purpose. Once settled on the designated dimension unit of stiffness, even though for an identical structural setting but one can always opt for different conventions, for instance, one shouldn’t be surprised at seeing that 1 kips/in is 1000 times stiffer than 1 lbs/in.

## 5.18 Lateral Stiffness – Standalone Crane Runway Girder

Whenever “studying” the subject of stiffness from a generalized “**simple bending**” viewpoint, the specified **DOF** – whether of linear or rotational nature – can only be associated with one of the **Principal Axes X** or **Y**, i.e. passing through the **elastic centroid**, it also means passing through the **shear center**, otherwise it is not “simple bending.”

Remember numerically speaking, **LOI** and **DOF** belong to two independent tribes; **LOI** is a group of scalars while **DOF** is a single vector. To begin the journey into a deeper territory, pick any **LOI** of interest then try answering, what happens if off from which the specified principal axis-based **DOF** does not pass through the **elastic centroid**? *Would that violate the simple bending decree*?

Clearly a few points we can take note from for this case:

- A load vector pointing into the vertical-**Y** direction would automatically take part in bending about the **X** axis and accompanied by rotation about **Z**
- likewise a vector pointing along lateral-**X** would automatically engage bending about **Y** and rotation about **Z** also
- Whether the **Z**-vector was drawn through the **elastic centroid** or not, the cross section is expected to deflect along **X** and/or **Y**

All seemed natural events; but one needs to pay extra attention to the consequence. Once the **Z**-rotation was brought onto the playing field, the structural behavior would no longer be *simple bending so that business is not as usual*;

> Regardless (1) which **DOF** was designated and (2) whichever **LOI** were paired up with, the arrival of **Z**-rotation would foul up the *simple bending*-based game plan big time
>
> It would sure turn up a notch more out of our mental responsiveness to some of us in order to grasp the broadened concept of stiffness − making it more confusing for time being − and would make the calculation of stiffness associated with the *non-simple-bending-earmarked* **DOF**/**LOI** that much more complicate, too

It doesn’t take a whole lot to follow along the “how” and “what” part of the subject to make sense of once all is said, heard and done with, but before doing anything further, the (not so) tricky matter is to identify correctly what/which **Z**-axis we were recognizing with. Would it pass through the **elastic centroid** or the **shear center**? Certainly the correct answer is **shear center** that should apply to both symmetrical and unsymmetrical sections.

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A linear **DOF** whether pointing along **X**, **Y** or **Z** always stir up our primary recognition, we take that as one-dimensional in nature; whichever the axis that is, it merely labels the general orientation aiming into the **3D** space, and nothing more.

In other words it does not make much sense to talk about stiffness until a certain specified "load" acting along a specified **DOF** was "pinned or anchored" at a specific **LOI**.

During the stiffness calculation, once locked in the **DOF** to the applicable **LOI**, it's time to take up to further details on how to deal with the *"probable presence" of torsion* on account of the latent <u>eccentricity</u> of **DOF** against the **elastic centroid** and/or **shear center**.

Herein if doing it the "more appropriate way," one would not start calculating **<u>Lateral Stiffness</u> until** formalizing the specification of **LOI** on **Crane Runway Girder's** term:

> The leading word "Lateral" merely identifies that the "Stiffness" application is intended for a load vector pointing along (not rotating about) the lateral **X-Degree Of Freedom**, implying that it calls for computation of **X**-deflection corresponds to the given $\mathbf{P_x}$ load vector, as simple as that
>
> Besides fixing up the load orientation or so beyond ordinary mind's eye, what needed still is the *exact target elevation* where $\mathbf{P_x}$ is applied at; for **CRG**, it is "always" at the *top of crane rail* – definitely not passing through the **elastic centroid** – there it fixed the **Y**-coordinate

Once finalized the **LOI's X/Y/Z**, say, *if we were to work out the stiffness value at the mid-span of a typical **CRG** due to $\mathbf{P_x}$ applied at the rail top then*, isn't it too easier said than done?

> Yes, but, before coming to conclusion, it pays to dig into what may happen from $\mathbf{P_x}$ being applied at some distance away from the supports, then take notice of what happens at the load point:
>
> > *Would it only deflect sideway along the **X**-axis matching the $\mathbf{P_x}$ load orientation?*
> > *Would it also rotate about a line parallel to the **Z**-axis through the **shear center**?*
>
> Easier to take care of the second question first; the answer is Yes, no dispute there, **Z**-rotation is always active
>
> Then to the first question; it would sound more affirmative by stating that "each and every node within the **X/Y**-profile would deflect not only along **X**-axis but would also deflect along **Y**-axis (*Reader to see the reason why*.) These happenings always apply whether the **CRG** profile is of symmetrical shaped or not

<u>In all **CRG** applications, simple bending may survive through willful assumption made in classrooms but it does not sustain in real life</u>; visualizing $\mathbf{P_x}$ being placed at the rail top over top flange, the fact to always remember, $\mathbf{P_x}$ "**rarely**" or more so "**never**" passes through the **shear center**,

> *In response to the application of $\mathbf{P_x}$, the girder section would undergo multi-dimensional movements: (1) rigid-body **X**-translation and (2) **Z**-rotation − **X**-translation is understood as the rigid-body linear **X**-offset against the "**elastic centroid**" while rotation is the angular **Z**-deformation pivoting about the "**shear center (SC)**"*

Rigid-body translation and rotation are two "normalized modes of deformation."

> Although in structural analysis the two modes of deformation could be decoupled into independent **DOF** for easy handling during the process, but the final state of **CRG** displacement at the load point would be the combined effects from "**initial X**-translation" and the "**subsequent Z**-rotation"
>
> Notice herein the use of two timing-sensitive adjectives, "**initial**" and "**subsequent**" purposely, simply to indicate the alleged "order" of events taken place one after another as in all **CRG's** standard structural responses to lateral thrust load

In the "mixed-mode deformation model," any tangential vector initiated from an angular rotation about the **Z**-axis could always be **resolved** into two linear translations through component projections conforming to any orthogonal set like our **X**- and **Y**-axes. Finally:

- The **resolved X**-projection (from rotation) would merge with the **initial** rigid-body **X**-translation into final **X**-side-sway and

- The **resolved Y**-projection would coerce every infinitesimal particle within the girder profile to play along with the vector orientation – moving either upward or downward

As result, it forces the **CRG** profile to exhibit a "side-stepping" move at the top flange rendering a "tilted" look. That explains why does an **X**-force ($\mathbf{P_x}$) offset from **SC** could induce both **X** and **Y** displacements. Sounds familiar and reasonable in theory but how to calculate the **initial X**-translation befits the next focus.

For symmetrical sections, the computation would involve no more than "one standard algebraic term." But it can get fairly knotty when dealing with **unsymmetrical sections**, as we need to invite the **principal axes** into the picture, the computation for which may entail these steps:

- Based on user-defined **X-Y** system, locate **elastic centroid**
- Calculate **principal angle** − orientation of the principal **X'** and **Y'** axes
- Calculate major and minor principal moment of inertia $\mathbf{I_x}$**'** and $\mathbf{I_y}$**'**
- Resolve the applied **X**-force ($\mathbf{P_x}$) into respective **X'** and **Y'** components, $\mathbf{P_x}$**'** and $\mathbf{P_y}$**'**
- Calculate **X'** and **Y'** deflections due to $\mathbf{P_x}$**'** and $\mathbf{P_y}$**'** using corresponding $\mathbf{I_x}$**'** and $\mathbf{I_y}$**'**
- Apply proper sign convention mandated by angular orientation of **principal angle**
- Combine **X'** and **Y'** deflections into a "resultant deflection vector"
- Calculate "**X**-translation" as linear projection from "resultant deflection vector" onto **X**-axis

Looks familiar?

> Notice that the simple phrase "**X**-translation" was emphasized to tell apart from the term "final **X**-side-sway." These (eight) steps on deflection calculation is one of the "standard flexural treatments" involving **unsymmetrical bending.** *It has nothing to do with the **Z**-rotation about the **shear center** as described in the "mixed mode deformation model."*

After all, there shouldn't be confusion from taking "unsymmetrical bending" for "unsymmetrical sections" for these are two different entities under engineering meaning.

---

What to be demonstrated next is proving how easily to get tangled up in **CRG** applications even for <u>symmetrical sectioned members</u> if not being watchful about what **shear center** could do to the structure.

The calculation of stiffness at mid-span of **I**-shapes due to unit lateral thrust $\mathbf{P_x}$ applied at top of rail is demonstrated in several "simple" examples hereinafter:

**Example 5.1**

<u>Given</u>: **CRG** W24X84 of 20' long, simply supported flexural boundary conditions-both ends, simply supported torsional boundary conditions-both ends. Rail depth $d_r$ = 6". E = 29000, G ≈ 11154.

<u>Required</u>: Stiffness at **Mid-span** along **X**-axis due to **X**-load applied at rail top.

<u>Solution</u>:

L = 20' = 240"
$P_x$ = Concentrated load applied at top of rail

W24X84 (**AISC**);

$d$ = Depth = 23.7”
$I_{yy}$ = 70.4 (not $I_{xx}$ for weak-axis bending)
$C_w$ = Warping constant = 12800
J = Torsional constant = 3.7

Since the principal axes match geometric axes for doubly symmetric profile,
∴ there is no need to resolve $P_x$ into X’, Y’ components.

Formula Ref: **Roark’s**
Applied load/torque at mid-span;

$\Delta_1$ = Deflection due to pure flexural bending about Y-axis from $P_x$ applied at mid-span
$= P_x L^3 / (48\ E\ I_{yy})$
$= P_x\ 240^3 / (48 * 29000 * 70.4)$
$= 0.141\ P_x$”

$d_0$ = Distance from rail top to shear center
$= d / 2 + d_r$
$= 23.7 / 2 + 6$
= 17.85”

$\beta$ = Torsion characteristic parameter
$= SQR(GJ/EC_w)$
$= SQR(11154 * 3.7 / 29000 / 12800)$
$= 0.01054$

$\beta L = 0.01054 * 240$
$= 2.5306$

$T_0$ = Torsion about Z-axis due to $P_x$ with $d_0$ offset from shear center
$= 17.85\ P_x$

$\theta$ = Rotation about Z-axis due to $T_0$
$= T_0\ (\beta L/2 - \tanh(\beta L/2)) / (2\ EC_w\beta^3)$
$= 17.85\ P_x * (2.5306 / 2 – \tanh(2.5306 / 2)) / (2 * 29000 * 12800 * 0.01054^3)$
$= 0.00848\ P_x$ rad

$\Delta_2$ = Translation as X-projection from Z-rotation
$= \theta * (d_0)$
$= 0.00848\ P_x * 17.85$
$= 0.151\ P_x$”

$\Delta$ = Total sidesway
$= \Delta_1 + \Delta_2$
$= 0.292\ P_x$

$K_x$ = X-translation stiffness at mid-span
= required $P_x$ to induce 1” X-deflection at the rail top
= 1 / 0.292
= 3.425 k/”

The implication: It takes 3.425 kips force applied at the girder's mid-span at top of the crane rail to deflect the girder laterally by 1" along **X**-direction

Now what if we calculate the stiffness at a **LOI** other than at **Mid-span**?

**Example 5.2**

Given: Same **CRG** conditions as given in **Example 5.1**

Required: At **one-third** span, calculate stiffness along **X**-axis due to **X**-load applied at rail top.

Solution:

Retaining all the variables, notations and parametric values as that for **Example 5.1**, we consider the general formulas (derivative per **Roark's**) which are applicable to **LOI** of all **z**-coordinates, ranging from 0" to 240", but for the numerical calculation at **one-third** span:

Let **z** = 80"

$\Delta_1 = P_x z^2 (L - z)^2 / (3\ E\ I_{yy}\ L)$
$= P_x\ 80^2 * (240 - 80)^2 / (3 * 29000 * 70.4 * 240)$
$= 0.1115\ P_x$"

$\theta = [T_0 / (EC_w\beta^3)] * [(1 - z / L) (\beta z) - \sinh(L - z) \sinh(\beta z) / \sinh(\beta L)]$
$= [17.85\ P_x / (29000 * 12800 * 0.01054^3)] *$ _
$[(1 - 80 / 240) (0.01054 * 80) - \sinh(240 - 80) \sinh(0.01054 * 80) / \sinh(0.01054 * 240)]$
$= 0.041068\ P_x * (0.562133 - 0.39595)$
$= 0.0068248\ P_x$ rad

$\Delta_2$ = Translation as X-projection from Z-rotation
$= \theta * (d_0)$
$= 0.0068248\ P_x * 17.85$
$= 0.1218\ P_x$"

$\Delta$ = Total sidesway
$= \Delta_1 + \Delta_2$
$= 0.2333\ P_x$"

$K_x = 1 / 0.2333$
$= 4.286$ k/"

Notice that both examples were for symmetrical girders whereas for unsymmetrical sectioned **CRG** the major difference, among others, is (1) the not so attention-grabbing locations of the **Elastic Centroid** and **Shear Center** and of course (2) the effect from unsymmetrical bending behavior.

For our **DOF** of interest fixed within the girder span, obviously the stiffness at mid-span is most flexible compared to that being elsewhere. Conclusion could now be summarized from the examples that stiffness along a particular **DOF** is dependent on the following parameters:

- Member boundary conditions
- Material properties and geometric attributes
- Member length and the pinpoint loading location

Therefore for a standalone member as demonstrated, the stiffness at a selected **LOI** is expressed as a single numerical value in absolute term, for instance along the **X**-direction, 5 kips per inch at point "A" or 500 kips per inch at point "B," etc.

However, things could become way more interesting by expanding our views beyond a standalone member into a set of members.

## 5.19 Lateral Stiffness – Coupling Crane Runway Girders

Looking into a typical service aisle peeking along the **Z**-fringes, there two standalone **CRGs** were spaced at **X**-distance apart; each girder would stay being independent until an Overhead Crane is charging into the service-bay. It follows from the predictable mechanical-structural interaction:

> At either end of the Crane Bridge Girder's **X**-extent where End Truck assembly was affixed to, under which, there were assorted number of Wheels set apart at various spacing along the **Z**–axis
>
> As crane wheels bear down against the Crane Rail, each **CRG** would be dispensed with complementary number of **Locations of Interest** (**LOI**) over its top flange; whereby all the as-built **LOIs** on both girders would stay at the same **Y**-elevation
>
> > Each **LOI** then matches its **Z**-coordinate in one-to-one correspondence with a unique **3-way** – *crane wheel-crane rail-**CRG** flange* – **track mark** of its own
>
> Respectively, the **X/Y** action-reaction exerting from each individual wheel would be taken at the top flange of relevant **LOI** on each girder

Soon as the runway system engaging two girders is subjected to lateral thrust load, the vast interest is in *how much* each girder can "capture" its own share of the *total lateral **X**-load sum* through interaction between the *Crane Bridge* and *both **CRGs.***

> For the two girders − as if communicating incidentally with each other owing to the presence of overhead crane strutting in between − since (1) they are in parallel with each other on separate **Z**-paths, and likely in some cases (2) the **LOIs** on the near side girder *may or may not* share, respectively, identical **Z**-coordinate with **LOIs** on the far side girder, Why?
>
> > The answer lies in whether or not if the two girders are of same length. Not only that, even though both girders are of identical length, but any dissimilarity in the geometric attributes, material property and boundary conditions, etc. can have an effect
>
> In any case, we could still calculate the lateral **X**-stiffness individually by whichever approach (similar to that as given in **Example 5.1** or **Example 5.2**) as seemed fit. Thereby the rendered **X**-stiffness values *may or may not be* identical and that depends on the girder geometry and a number of additional attributes inherent in each girder

Just visualize the two **Z**-members (**CRGs**) were "pseudo-united" through a spontaneous event of *Crane Bridge-End Truck-Crane Wheel-Crane Rail-Crane Girder Flange* engagement at each other's relevant **LOI** having a common **Y**- and **Z**-coordinate suite, in turn the two members would have to *act and react* in unison in response to the strutting **X**-action disbursed from the crane bridge assembly:

> Under the circumstance, it would be more meaningful to examine the stiffness in relative terms instead of using exact values, especially as both members are in contention for allotment of structural attention, which is yet to be divided up fairly and squarely
>
> As in the classic "flexural moment distribution" procedure,

For which we need to figure out the **distribution factor** for each member at the **LOI** based on the "relative stiffness," – *<u>at this point</u> it is much easier to comprehend if merely bringing out the catchphrase* "relative stiffness" *than actually obtaining the true numerical values, and we shall see why* − and thereby we are able to tell, for example, 500 kips per inch is 100 times stiffer than 5 kips per inch

<u>In the simplest case,</u>

One could grasp the straight fact without much deliberating (through lengthy calculations) provided the girders on either side − the **near side** and the **far side** − of the crane aisle are of:

- Same boundary conditions
- Same support connection detailing features
- Same matching profile configuration and section properties from end to end and
- Same length

By all means, the given geometric-physical configuration in both girders would be <u>identical</u> with respect to each and every corresponding axis-of-reference

Should this setup is maintained in both global and local connection detailing levels then all the key section profile parameters/properties would be respectively <u>identical</u> as well

Obviously considering the relative stiffness' worth alone, any **DOF** trending at any focal point along the **Z** span for both girders would have been one-to-one correspondingly <u>identical</u>, too, and therefore the "apportioning ratio" for general-purposed distribution (of lateral thrust load, for instance) should be 50:50 or 1:1 if judging only from a <u>pure rigid-body static load application</u> point of view, that is weighted in without probing into the exact stiffness value of either girder

Certainly, 50:50 ratio is a justifiable lateral load-sharing situation − with or without stiffness calculation − in this case when the near side girder is identical to the far side girder comparing all as-built features

However, **nothing is ever perfect**, because most **CRGs** live and work under a very dynamic and more often not well-disposed environment where an ideal 50:50 rationing ratio is hard to maintain for long or hard to come by after all, *the Authors recommend to use a 60:40 distribution, in other words to use 60% of total lateral load as minimum for each girder <u>to cover any uncertainty</u>*

Consider some extreme situations, there it is not unusual that (1) the trolley was "constantly" hitting the bumper with a "favorable" full slam on one end, in a way bumping on one of the girder flanges or, look closely and realize that (2) somewhere the rail clip bolt or weld – and/or thrust plate bolt or weld − could be sheared or certain weld could be cracked rendering an unbalanced stiffness ratio, etc. so what distribution should we use then, *70:30, 80:20, 90:10 or 100:0?*

**Nothing is ever perfect**; in other words, being stiff or flexible maybe a cut-and-dry matter on paper but not necessarily that positive definite in real life, and we should never assume everything is perfect in any moment.  See the issues that can come off in real **CRG** life?

Ever speculating what happens when we came across those situations?

What if the "**far side**" girder and the "**near side**" girder were **differed in every aspect** except that they are of the same length?

For comparison, if both girders were scanned methodically along the span from "**0** to **L**" on corresponding **LOI** of common non-zero **z**-coordinate, a few technical-numerical specifics noted as follow should catch our further interest:

- The key torsion-based properties, namely $\mathbf{I_{yy}}$, $\beta$ and $\mathbf{C_w}$ – and perhaps $\mathbf{d_0}$ (*distance from rail top to **shear center***) − in both members are **always** one-to-one correspondingly *different*

- *Interestingly but no surprise herein when considering the crane is on the move*, notice the flexural translation along **X** axis is **always** a nonlinear function of **z**, **L** and **L - z** while the torsional rotation about shear center is **always** a function of **z**, **L**, **L - z**, $\beta$**z**, $\beta$**L** taken selectively as parameters associated with specific Hyperbolic Functions

- The lateral **X**-displacement value is **always** the sum of two parts: A flexural term and an **X**-component resolved from torsional rotation

- Having identical boundary conditions or not, the relative stiffness ratio would **never** be as straightforward as 50:50 or 1:1

Despite the fact both girders are of same length, but, with so many unfavorable characteristics stuck under those so-identified **always** conditions as pointed out from above,

> One should never anticipate that the "relative stiffness ratio" would remain constant when tracing from one **LOI** to another **LOI** along the **z**-axis; and therefore the lateral thrust load "apportioning ratio" distribution would **always** be "lopsided" and would vary from node to node along the **Z**-axis, although how lopsided and how may the gradient of variation look like remain to be seen yet

Again, if this is not a good topic for further **R&D** then what is it? See the dilemma, academia?

Now contemplating the **most unusual setup**:

What if the girders on both sides of the service aisle were dissimilar in every aspect – geometric and detailing parameters – and even the member length?

One can dig deeper based on a real life example, given conditions that:

- The near side girder is 20' long − a standalone rolled **W**-shape (doubly- symmetric) with a cap channel on top and is free from any other major "structural" attachments

- The far side girder:

  1. Is also a rolled **W**-shape but of different designation
  2. Has thrust plate on inner side of the top flange
  3. Is 40' long (allow for truck/shipping/maintenance access from underneath)

  The fact:

  Whenever an **I**-shape has a thrust plate "connected" on one side, the "combined" section profile becomes unsymmetrical, can't challenge that

Without doing an actual calculation hereinafter, we could play along with several "**lateral thrust loading**" scenarios for this **most unusual setup** and see if anyone could get a firm grip on how to resolve the stiffness issue:

(a) Placing the overhead crane centered about the mid-span of 20-footer

For the 20-footer:

With the load sourced at its most flexible spot, it's fairly straightforward to compute the stiffness in ways as in **Example 5.1** − from looking up **W**-shape section properties in **AISC** and using formulas per **Roark's**.

For the 40-footer:

The calculation may involve following steps:

- For the built-up (unsymmetrical) section, calculate the *strong-axis and weak-axis moment of inertia for both "user-" and "principal axis-" based coordinate systems, then shear center coordinates, then warping constant and torsion constant*, etc.

- Because the load is now acting at the quarter span, one needs to resolve the weak-axis translation per procedure applicable to **unsymmetrical bending**

- Calculate rotation at the quarter span using generic formulas per **Example 5.2** (a bit more complex than load being applied at mid-span)

- Calculate concluding "**X**-sidesway" by combining the flexural and torsion effects

(b) Placing the crane centered about the mid-span of the 40-footer

For the 20-footer:

Obviously the "**resultant** of loads" is pointing right into its building column support; theoretically there is neither **X**-lateral deformation nor **Z**-rotation from a girder end's supporting point of view,

> For that humble reason, supposedly, should the stiffness (for whichever **DOF** of interest) be qualified as **infinity** then?

By instinct, if observing the lateral load **resultant** (reaction) as a concentrated natural object without going into further details then, "yes" and **infinity** would be a reasonable answer, but,

> T*he true answer is no*

One can start from the simplest fact (not an assumption):

> Consider the basic outfit that is more than likely shared among all overhead cranes, there are at least two wheels under each end truck, clearly we can see what happens if only the end truck is centered about the building column;
>
> If given exactly two wheels then, they would have been straddling over two adjacent girders sharing a common column support
>
> > Obviously one of these two wheels would be settled over the 20-footer in focus at certain **Z**-coordinate measured from the column, and the other wheel would be landed on the adjacent girder at certain **Z**-coordinate measured at the opposite side of the column
>
> Then, in the stiffness calculation, should we consider the effect due to "offset **Z**-distance from the support" at each individual wheel location although the answer may be "No"?
>
> (Dare someone say "Yes" and if so then explain why) But how do we determine the relative contribution of lateral translation and rotation from each wheel? – So the stiffness

of the 20-footer is not **infinity** after all; or still is? Consider this is a good homework problem

For the 40-footer:

Among all probable scenarios, there are only two major cases to consider:

- Load source is **pulling** one way along the crane bridge **X**-direction **away** from the 20-footer on the opposite side

- Load source is **pushing** into the other way along the crane bridge **X**-direction **towards** the 20-footer

Further questioning in **either** case:

- How many percent of lateral load is absorbed by the 40-footer? Is it 0 or 100% or of some other values?
- Is 100% going into/away from the supporting column at the 20-footer?
- More questions/answers?

(c) Placing the crane arbitrarily at anywhere else

*For either the 20-footer or 40-footer, would it get even more complex and more confusing?*

Obviously there doesn't seem a rational way now available to apportion the **Lateral Thrust Load** in this situation. No matter where the crane travels to or from any location, we are challenged from keeping track of (1) the "variable load source placement" and (2) the "variable stiffness ratio" along the full 40-foot length.

The bottom line:

As the crane traverses back and forth along the global **Z**-axis, the ratio (percentage) for distributing the lateral thrust load into each girder changes nonlinearly with respect to change in the **Z**-coordinate, see the time-lag-dependent complication?

Visibly it is not a continuous function in mathematical sense, simply because there is only *one 40-footer over two columns versus two 20-footers involving three columns*

Should this warrant a full-blown **R&D** effort to clear up the hidden vagueness?

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Further deliberation:

**Examples 5.1 and 5.2** utilized the formula $\Delta$ = "total lateral side-sway" combining the rigid-body **x**-movement and the component **x**-projection per rotation about **shear center** = $\Delta_1 + \Delta_2$ as the basis to calculate girder stiffness. Implicitly it was on an assumption that the rail does not at all participate in the act

But on deeper thought or simply of pure curiosity, unless the rail is being fixed in place "continuously" otherwise it would have to "float" passively along with or against the girder's lateral movement; especially when the rail floats locally due to loose clip(s) or missing clip(s)

Before reaching the girder flange, "lateral load" starts out as a point load soon as the "wheel" is in contact with the "rail." Normally rail spans intermittently between clips at **Z**-intervals prescribed by the types of clamping device being installed. Comparable to how girder behave, the rail counterpart $\Delta_{1r}$ is due to the initial **X**-translation while $\Delta_{2r}$ is due to the subsequent **Z**-rotation

So then **how about considering the mixed translation** $\Delta = \Delta_1 + \Delta_2 + \Delta_{1r} + \Delta_{2r}$**?** In that, respectively, $\Delta_1$ **and** $\Delta_2$ (= Girder$\Delta$) were the translations of girder, $\Delta_{1r}$ **and** $\Delta_{2r}$ (= Rail$\Delta$) were the translations of rail

The cross section of a crane rail is of singly symmetric profile when newly installed. Once the rail wore out during service over time, the section properties (area and moment of inertia) could be reduced substantially. In consequence the section becomes unsymmetrical. This should affect the $\Delta_1 + \Delta_2 + \Delta_{1r} + \Delta_{2r}$ sum even though the attributes of girder on both ends of crane aisle might be unaffected from a global perspective

In fact anyone can hold an opinion suggesting that the influence from rail is negligible. But the more convincing approach is looking into some particulars before drawing such conclusion

The schematic for calculating "Rail$\Delta$" is similar to that for $\Delta_1$ and $\Delta_2$ except that we need to apply (reduced) section properties from a gouged or worn rail section and deal with a number of other technical issues that may involve:

- Properties including moment of inertia for **X/Y** axes and principal axes, torsion constant, shear center, warping constant, $\beta$ value, etc.

- Vector resolution for unsymmetrical bending
- Rigid body rotation and rail neck rotation due to cantilever effect
- Effect of friction resistance at rail base
- Attenuation effect compensating to variation of rail clip spacing
- Effect from rail float and/or bearing against the supporting clips, etc.

Whether in theory or by wild imagination, it doesn't take much for Readers to bring on all that may apply. Figuring out the weak axis moment of inertia for a worn rail is already a chore in itself, let alone taking care of other logical/numerical uncertainties. A reasonable approach should be: Unless it has been proven otherwise, the effect from $\Delta_{1r}$ and $\Delta_{1r}$ may not be negligible after all

**What if considering "Rail$\Delta$" and "Girder$\Delta$" separately?**

We could start out from recognizing the timing sequence of events and notice in that "Rail$\Delta$" always takes place before "Girder$\Delta$" does

Obviously the girder should not be moving if the rail does not deflect. Besides that, the lateral load transferring mechanism is a dynamic episode. When the rail realizes the full extent of "Rail$\Delta$," it instigates a damping effect. We may consider that as if a lateral spring is being compressed that brings on energy absorption. The resulting force into the girder flange would likely be reduced to an amount much less than the applied load to the rail

Whether understanding or misunderstanding the intent is a different matter; but does this sound more like a new homework problem for more **R&D**?