Open Sectioned Crane Runway Girders with Arbitrary Profile Geometry

Chapter 1 – Introduction

John Fong (馮永康) . Bill Vanni
Structural Design Corporation
1133 Claridge Dr., Libertyville, IL 60048

The simple phrase "**Crane Girder**" when generalized in most Industrial Facilities could stand for disparate objects fitting specific area of focus linking unambiguous Mill Functions. For this very reason while not being explicit in certain contexts, the matching expression could be referring to (1) the component of an **Overhead Crane Bridge Assembly** or (2) a portion of the **Crane Runway Support Structure**.

1.1 What is Our Focus?

Under regular setup in most Mill Facilities, **Crane Bridge Assembly** and **Crane Runway Support Structure** are collaborated to achieve load transporting and load supporting purposes.

When identified more closely by their physical roles:

> Visibly, Crane Bridge Girder is an important share of a specific type of overhead crane mechanism driven along the runway whereas Crane Runway Girder is a structural constituent that stays immobile while supporting a specific segment of runway system that overhead cranes run on

If pinned down exclusively from an Engineering Design Viewpoint then:

> Crane Bridge Girder is deliberated in accordance with Mechanical Operation Requirements while Crane Runway Girder is outfitted in line with Structural Supporting Function Requirements aptly configured under the Strength Design Provisions

Knowing the distinction and much as to avoid misinterpretation of our objective, it is important to set the focus upfront:

> This **Series** were meant for those interested in the Structural Engineering Aspect of the **Open Sectioned Crane Runway Girders** having either **Symmetrical** or **Unsymmetrical Section Profile**, or in mind's eye in what could be broadened to other applications involving *open-sectioned stick-like* members with **Arbitrary Profile Geometry**

1.2 Where Do We Start?

Knowing from this point on there would be more **Chapters** with more pages to follow in the **Series**, but where do we start?

> Sometimes it's better off starting from scratch through a universal conception, in which:
>
> - Provided all "Structural Organisms" were characterized on equal ranks and bases − regardless to their appearance, how they were loaded and what they were intended for, etc., and
>
> - For all given structures provided in fulfilling our Structural Engineering Responsibility on principle as to questing for the highest Engineering Quality per se;

And then realizing that there wasn't much difference to tell from one structural class to another – a fair game to start from scratch

Judging by the plainness in appearance alone, *stick-like* **Open Sectioned Crane Runway Girder** (**CRG**) may not be appreciated as much into a special class of its own. Although being *stick-like* is rather simple and common, yet into further detail no matter whichever classification they belong to, the dealing with **CRG** is never a normal pastime.

While facing up to specific **CRG** issues, unless by coming through beyond the superficial equal-ranks-and-bases notion and advancing into what truly counts as to preserving the structure's *in-service longevity*, it then became much more evident that the engineering/technical treatment to **Open Sectioned CRG with Unsymmetrical Section** does take up bountiful *no-nonsense analytical-design-detailing proficiency* with very little wiggle room for *negotiation.*

Still, within the domain of "Structural Organisms" whether of **CRG** or **non-CRG** class, how much would an individual pay due **respect** (or not at all) as warranted to any specific Structural Engineering Subject would highly depend on (1) one's area of concern, (2) one's depth of awareness and (3) one's reception or admiration to the subject of interest, etc. But by whichever stance acquired, one might take on different position at different occasion as seen fit to responsibility entrusted to the individual, for instance:

- Firstly, from a **generic** fundamental engineering/design treatment **viewpoint**:

  Per Normal Structural Engineering Principles that most of us were familiar with:
  There should be no variance in our *professional attitude* toward **CRG** in the Mills − Material Handling Facilities or the like − than that toward other varieties of linear stick-like member as far as strength provision for structures' supporting function goes

- Next, from a **contractual viewpoint**:

  Any structure as engineered, as fabricated and as built, should meet all commitments as obliged to all as stipulated in the material/design/construction specifications and should last as long lasting as anticipated

- Lastly but not the least, from a **legal viewpoint**:

  By all means the responsible Engineer of Record should be accountable for the design that should have attended to all given load scenarios − exhausting all probable ***loading-unloading*** events and their due combinations in due course considering all due extreme load patterns and magnitudes acting along all due load senses, etc.

Yet if we were to accomplish every single assignment communal from all perspectives as pointed out above, big and small, onward to covering all stages combining pre-construction (*design engineering*) and post-construction (*upkeep maintenance*) matters then without a doubt, **CRG** should stand out above most ranks of structural member type – conceivably owing to meeting **engineering-technical challenges** in order for the *as-installed* **CRG** to perform well in service *afterward*, day after day, years after years for as long-lasting *as expected*.

For those not so convinced of what may come as unavoidable situations in the thick of it all, just wait and see; here are some sample challenging tasks of Specific Structural Engineering Interest, which for sure should demand exceeding efforts from us whenever facing up to a typical **Crane Runway Girder**:

- The comprehension of its **tricky** load-response behaviors
- The handling of **complex** states of stress of wide-ranging varieties and patterns

- The fulfillment of **all-inclusive** design commitment taking in especially the often disregarded, mishandled or misunderstood discipline on provisions against metal fatigue (only then the structure could last for as long as anticipated)

- The development of a **practical** data management scheme suitable for intermittent design debugging and review, design optimization and for final engineering document presentation/preservation purposes

- The implementation of an **effective** engineering-themed data analysis system and data bookkeeping process geared for all practical engineering purposes, and so forth

More often than not:

While tending to those rudimentary technical matters during *initial* design phase, too many Structural Engineers were preoccupied too exceedingly in muting down the clamors from **nominal** stress-and-strength requirement – becoming fixation of spending lowest possible effort in all engineering activities, still that seems to work most of times

Being so wrapped up in extra frugal and inflexible engineering mentality for too long, some would not contemplate on **CRGs'** term what could happen beyond the *initial* design phase or never caught on to consider the in-service *upkeep-maintenance and serviceability* issues, all that, plus some other unseen out-of-norm happenings that duly had control over the ultimate structural performance

Of characteristic nature, the timing of occurrences of these in-service issues/events and the seriousness of that were unpredictable ahead of the end games anyhow, hence these **CRG**-unique matters of concern were overlooked – thus *leading to major takeaway from anticipated structural performance*

Of typical setup;

Engineering endeavors in time-honored Architectural Engineering (**AE**) Design Session often "end" at the signing-off of structure's design phase**, normally** preceding or barely into the fabrication/construction phase

Such setup whether by contract, by chance or sometimes by choice, but that was the long-established norm, unless so commissioned otherwise. Then going with the tradition of no big surprise, nearly all *normal* engineering visions under *normal* project scopes in *normal* **AE** trades were not mandated to further into the **maintenance** matters − which took place only during the structure's in-service phase – *a logical reason as to why upkeep-maintenance and serviceability issues were not in their books*

One wonders:

Just what in the nutshell of *upkeep-maintenance and serviceability* concerns that was more important than **nominal** stress and strength?

First of all,
Taking home a minimum passing grade in structural serviceability is a forever-and-ever matter. The criterion for typical structure to earn such "lifelong status" could be boiled down to *scoring as high as possible* continuously in both of these categories: **durability** and **stability**

Secondly,
A reality came about quite commonplace following the design phase into construction phase: Structures with "inherent strength against instability" were well designed for conditions bounding all-inclusive effects from all due loading-unloading scenarios in the first place; yet it takes much

more "engineering thoughtfulness" in the handling for structures (1) to sustain and (2) to remain being stable and healthy into/beyond their due term

Finally,
To structures scoring high throughout in-service phase, ultimately, **durability** comes from adequate strength against **both metal yielding and metal fatigue** whereas **stability** comes down to adequate strength against **both global and local instability**

By reasonable deduction outside of the confine of stress and strength, if "serviceability quality" were articulated as some tangible substance that must be "enumerated" with reliable accuracy then, the treatment to **CRG** should at least entail (1) provisions, (2) upkeep and (3) evaluations of structural deformation data throughout all passing stages, respectively involving:

- The pre-construction phase: Estimating **realistically** and **accurately** the deformed shape(s) of structure from qualified engineering efforts

  *Of engineering awareness concerning serviceability, one should readily recognize the fact that deformed shape(s) of **CRG** normally conforms to **all six degrees of freedom** even though that seemed as if some unpretentious displacement per instantaneous perception*

- The post-construction phase: Confirming **sensibly** and **accurately** the permanent structural deformation, if any, from qualified field observations

  Therefore in making better sense to the Facility Owners, the overall structural upkeep-maintenance/serviceability performance through evaluation of records compiled from inspection/survey could only be confirmed in the post-construction phase

Sure indeed, as for **CRG** Engineering Concerns beyond acknowledging that (1) the structural behavior is *tricky* and (2) the handling of the mess is *complex*, etc., there is still a big unknown:

Could there be other technical traps abound or treasures (or troubles) hidden?

By any means, anyone could have a quick-witted answer on reflection or be humble and not so hastened until probing much deeper later on. But prior to doing that at any pace, keep in mind **the subject matter**: ***CRG with unsymmetrical section***, on which a few reality checks would follow:

- **The Bygone**: Over time, girders configured with the said profile geometry were very much "in style" in various Mill Structures and Material-handling Facilities

- **The Present**: These structures so configured were popular not only in the older facilities but also well represented in some of the newer ones as well

- **The Fantasy**: Simply by their wide-ranging reception, it would be fair and logical for those "unsuspected believers" among Structural Engineers to "believe" that the Industry at large should have accumulated plenty of know-hows on their structural behaviors, their qualification procedures and what it takes in fulfilling the functional requirements, etc.

All of the said reality checks sounded authentic enough. Not surprisingly but to the contrary of our expectation at **present** if merely (not) relying on the **bygone** and our **fantasy**. It seemed universally barren among many published Textbooks, Classic and Modern Standards, Codes and Design Guides, etc. that the topic on structural members with unsymmetrical section was very limited − either cut short or given coverage rather ambiguously and none were substantial for practical purposes

On realizing further:

- **The Forewarning**: What could be the worst part among all open books on the **CRG** subject?

  Depending on what issues were in focus, whichever and however the knowledge was applied to specific design matters, one needs to be on toes to avoid trusting falsely vetted information out there − some could be incomplete, misleading, unjustified or plain wrong

- **The Reality**:

  Be right or wrong it is much more difficult than it appears as to breaking into the subject of this enormity without proper lead-in from a dependable source

- **The Question**: Again, Where Do We Start?

As of this writing, there seemed too many/too few disorganized entrances and/or exits for getting on/off the **CRG** Engineering Information Highways and Byways.

Nevertheless, to begin appreciating what it takes to arrive at where we really wanted to be in the end, whether on/for technical or practical reasons and if truly defying a convenient point of entry into the **CRG** mainstay then:

> How about riding along a *technical itinerary* straight into the very basic design requirements – one that should be the most straightforward among all − **Flexure**?
>
> While focusing on **Flexure**, in addition to looking after the serviceability and stability issues, Modern Steel Design Codes implied that Steel Members Designed for **Flexural influence** should at least be evaluated for these basic modes of failure:
>
> - **Material yielding**
> - **Local buckling**
> - **Lateral tosional buckling**
>
> These **standard** evaluation criteria should have been appropriate to all member classes to start − and naturally that should have included **Crane Runway Girders** as all of us expected − unless excluded explicitly, of course

In solving Structural Engineering problems, it is much easier to score points (in terms of times and efforts spent) off everything being "pure standard" – attributes as given or as assumed − than anything infused with tidbits of non-standard element. One could easily draw up some not-so-standard circumstances hereinafter from any or combination thereof as follows:

> - *Non-**I**-shaped structural members − having unsymmetrical profile (excluding angles, tees, pipes, tubes and those of closed section in general)*
>
> - *Structures subject to repetitive loading-unloading live load series − to be designed in compliance with fatigue resistance mandate*
>
> - *Structural members experiencing uncharacteristic loads − inducing warping-related stresses*
>
> All seemed targeting at **unsymmetrical sectioned CRG**, which fit in with each and every **non-standard** element there is − imagine being assigned to deal with such atypical situations, it must be more than embarrassed if we were so technically ignorant of the evidences or so ill-equipped to qualify/certify the adequacy of structures so configured − the awkward question:

> *Aren't there any exceptions or extensions to the as-established **standard** qualification requirements formulated specifically for dealing with as-acquired **non-standard** circumstances?*

There is no simple answer to it at this point; but no matter, there should be an easier way out from the flip side − *provided knowing where to start.*

Although it's a long way through technical means to confirm if indeed there were exceptions or extensions applicable to the structural qualification of **CRG**, but to really get to the bottom of this it might be more practical to assimilate what were the generic must-dos and must-not first by simply combing through the most current Code-sponsored design provisions, and then while scanning through each and every page and chapter; we need to remind ourselves once more that **CRG with unsymmetrical section** is the main focus here – *so is there a problem?*

1.3 What Seems To Be The Problem?

Let's find out if there is;

Following the appreciation of latest **AISC** criteria (*since the green book edition as of this writing*) as to meeting provision per Section **F12** (UNSYMMETRICAL SHAPES,) Section **G2** (MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS) and/or Section **H3.3** (MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE) as outlined in one of the (most) recent **ANSI/AISC 360 (AISC)** on treatment to structural members with unsymmetrical section, then ask:

- First off, should one agree that the stipulation is adequate and sufficient for practical application involving **CRG** with **unsymmetrical section**?

- Were there sufficient guidance?
- Or don't see any problem?

As **AISC** stated plainly:

> In the opening paragraph of **Chapter F** that it "… *applies to **simple bending about one principal axis**. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports.*"
>
> Other comparable restrictions related to shear stress/strength provisions and the design requirement of web stiffeners, etc. were also declared in the opening paragraph of **Chapter G**
>
> Very thoughtful restrictions as stated, in fact there is nothing wrong with these restrictions;
>
> > Notice that the constraint of "**loads in web**" so as affirmed at each applicable Chapter's opening paragraph does cast a solid setback in the Engineering of **CRG** at the moment, and (likely) thereafter if continuing that unfavorable trend for practical use

Then take a closer look at all those ongoing **CRG**-related engineering ways and means at large, think hard:

> All of which were truly correct and helpful?
> Is anything missing?
> Could some of the so-call truth be *ruled* deficient if not all wrong?

Never mind the *verdicts* for now as all details would come to light much later, but before getting there, ever wonder where we came from?

While judging with a fair mindset on what were offered in one of the most recent Editions of **AISC** as of this writing contrasting the scant coverage from those related sections and/or subsections given (or not given) in the previous Code Editions, one should praise that these modern updates were indeed well thought out advices

All in all the Code Committee(s) at least for the time being as of this writing − but only up to a point though − did clarify much more clearly than ever that the stipulation is limited to what it proclaims with verbiage as is so that the Practitioners should not misunderstand or misuse the Chapter(s) intent for any conditions not (yet) **officially** endorsed therein, or else we as "*practical engineering design information buyers*" need to be aware of what sold or bought

But what a downer from further digging into the reality that:

Despite all the constructive advancements in addition to those fundamental provisions being put forward in the latest **AISC** as of this writing, one could still come across several compilations of very "iffy-sounding" advisory material given rather judiciously amid the texts

Those words/texts, on one hand, were beneficial if taken as informative recommendations, but on the other hand per same perusing session; the invariable advice could be nerve-racking to many Average Engineers as if hint of dissuasion if not warning. Various dispositions from interpreting the same texts could set us up into either an obliging comfort or an overwhelmed daunting state of mind. Thereby whether abiding by those remarks carefully or doing the otherwise becomes debatable depending on personal judgement

Hence quite likely, whatever the lasting impression one might unravel from those words (of caution or encouragement) verbatim by the Code statements without unabridged dress rehearsal(s) would always seem conjecturing or technically hazy at best

But taken merely on literal connotation, deciphering text messages or knowledge of words with *built-in uncertainty* into serious decision of what should or shouldn't be ruling a specific structural design topic and then put it into practice could lead to *serious* <u>*miscalculation*</u>. Not exactly the same but only in concept, the situation could be likened to <u>*misjudging*</u> the sharpness of a carving knife merely by its pretense to make a cut; the consequence could be very detached from truth − the difference of realization is in the timing; it might be instantaneous or might take a while

Take stock of what have we veiled in our **CRG** *engineering professional assets*:

Our assets in terms of knowledge or credentials, whether acquired through profound experience, collection of references, examples, rules per Codes and Standards, etc., are worthless and won't do any good being left on shelf, drawers or in our memory had that not been vetted through tried and true tryouts. Or else it would only be meaningful when "**unsymmetrical sectioned CRG**" comes under a commissioned engineering project calling for implementation utilizing what we "owned"

How comfortable one feels from taking on "unsymmetrical sections" differs with the depth of knowledge and experience one has − just picture the uneasy feelings from trying our fresh hands on such assignment with unchecked know-how − it could be as naive as if throwing a dart not knowing <u>beforehand</u> where it hits that could be lucky if no harm done <u>in the interim</u>, or be seriously wrong <u>afterward</u> learning the hard way from cruel legal lesson or experience defending to the jury of which edge could be used to make either a fine cut or a dull cut, and the like

Still rather gloomy to bring to light:

For unsymmetrical sectioned **CRG**, it seemed those technical vagueness (Code stipulation) would not help much to ones already caught up in the messy muddle being stuck with must-do must-win project ventures engaging mostly deficient/distressed **CRG** in command of much more diverse engineering attentions

And probably for ages already, we were ever more in the thirst for much better and much broader advice on the specific topic especially since these catch phrases ***Crane Girder***, ***Torsion*** and ***Unsymmetrical Section*** were formally welcomed into the recent Codes and many Design Guides.

Now whether veering on course or off course from wherever the Codes stand is not the issue, but the cruel realities we must face − if not aplenty already − are some of the inherent situations especially the "Loading Dilemmas" typical to almost all **CRG** whether or not with <u>unsymmetrical sections</u> to content with:

- *The applied **loads** rarely (or never) stay in a plane parallel to the principal axes*
- *Let alone of **load resultants** passing through the **shear center***

What humbles us even further if we were to strategize our "bracing" design scheme following the **Code** recommendations toward a high-grade product for good practices' sake; but then:

- *How could each and every **CRG** intended for a variety of Mill functions be effectively and feasibly restrained from twisting at each and every probable load point?*

- *Meanwhile how to deal with the fact that the crane (more than likely having multiple wheel load points on a single girder span) is regularly in motion during typical operations?*

Whether in theory or in practice, the flexural behavior of **unsymmetrical-sectioned** member − **CRG** or **non**-**CRG** − hardly ever fits in the narrow category of "simple bending" all because of the unavoidable charging from torsion. Now **what seems to be our problem**?

- Does it mean that "some of" the Code sections would still be <u>relevant</u>?
- Or be the Code sections applicable <u>only</u> to lateral-supported or torsion-free **CRG**?

- Conversely, could anyone conclude that: Since torsion by functioning nature is always a part of **CRG** loading characteristics therefore automatically the Code would not apply (or would not be helpful) if the lateral support criteria were <u>not met</u>?

- Or since torsion is not an out-of-the-ordinary phenomenon, should there be any addenda or a special Chapter for torsion alone or an expansion of **Chapter F** or **Chapter H** covering the combination of flexure and torsion?

Agreed? Disagreed?

1.4 Being More Confused? Having More Questions?

Again, depending on the individual's background and experience, each Structural Engineer or Engineering Company − be they from one of the Consultants, Fabricators, Contractors or our Clients, etc. of various positions and interests − <u>may have</u> radical differences in their opinions, which in turn lead to varying degree of awareness or ignorance on the subject. Therefore it should be of no bigger surprise seeing variations of questions, answers, interpretations, misinterpretations and ways and means, etc. regarding the treatment to **CRG**.

But for focusing on the big picture's sake, all of us should have come to a common ground in asking common and/or uncommon but sensible questions on what to do (or not to do) with specific issues without further evasions, because problems won't be solved if various factions were navigating in different directions or holding different grounds.

Went beyond what already covered in the governing **Codes** and so-called **Design Guides** of relevance dated back then up to present time, **there still seemed more misses than hits** on resolving certain issues.

To some, shortage of thorough directive or resolution to key design issues makes them feel like being driven into technical left-behind or nowhere land where these unsymmetrical sectioned members were dispersed; then wouldn't it be more constructive for those (among us) already caught up in the state of technical confusion to come through on their (our) own behalf?

As pointed out earlier, unsymmetrical sectioned **CRG** fits in a non-standard class of its own, which needed non-standard out-of-ordinary engineering attention; and yet comes to minding of their "in-service longevity" there isn't much "Official Rules" to follow or "True Engineering Help" in that regard

On such hot subject for now, it's still long way up to the Technical Summit. To fetch much needed Technical Amenities that we don't have in our Unincorporated Technical Subdivision, there left is our own enthusiastic pursuit based merely on engineering common sense, determination and mindset, etc. Would it be technically smart or plain folly to take on the **CRG Drill** on our own passion? Let's find out

Seriously then, **where do we start**?

Our mounting technical sorrows and pains can only be alleviated if only we build on prescriptive measure plain and simple. For that instead of going too hard too deep too soon, we may start making the turn right back to the Flexure Design Requirements searching for practical answer to a somewhat puzzling yet very straightforward question on **CRG** qualification:

*In the normal design drill against non-fatigue failure, is it **OK** to qualify **CRG** solely on basis of **material yield limit**?*

The inquiry could be of different construct for similar intent; yet clearly it is in dire need of a wishful "Yes" response. Imagine that, should the answer be in our favor then it would sure make every Structural Engineer's life much easier. And how nice should this be genuine, but then:

- *Could the same favorable answer be equally applicable to **CRG** of both symmetrical and unsymmetrical section profiles?*

- *And how about giving the same **OK** not only for **CRG** but also for **non-CRG** structures?*

Before nodding our heads we must identify with the fact that such technical favoritism is not at all "free of charge" without serious explanation of what is and whatnot; certainly better if all justifications were put in plain words widely verifiable from the current knowledge base at large.

1.5 What Is In Our Bucket of Knowledge?

And this would be a fine moment letting in unsolicited commentary on the subject of interest. To agree or not with any specific comments appearing in this Series of Articles is really not that important. What is important is why doing it. Why?

*No matter whether if most Structural Engineers accept or realize, the biggest fallacy in treating **unsymmetrical sectioned CRG** had been to keep borrowing (applying) some of the conventional (or unconventional) wisdoms that were inappropriate or inadequate to **CRG***

Knowing that any adversity or discontent from what advocated herein could ripple further beyond the local confine of **CRG**, but we should question ourselves:

*Is it **OK** to carry on with the same old engineering tactics based on limited bucketful of current (or bygone) **CRG** knowledgebase, or if so **OK**, should such "debatable scheme" be thought through twice or thrice the next time over before having another run with **unsymmetrical sectioned CRG**?*

Practical Engineering Knowledge amasses on solid experience. No matter how little or how abundant knowledge we gained from past experience, it could always twitch our sentiment on or off on unexpected impulse. It may make perfect sense to some of us as to describing what a "bucketful of **CRG** knowledge" really is in such a manner assuming that "knowledge" could be quantified:

> *Knowledge-wise, humbly the more (or less) someone thinks had gotten the hang of treating **CRG**, the more intimidating one becomes for fear of the bucket being half full yet being half empty at the same time, or plain feeling chancy*
>
> Ever felt like walking in astray through a maze of alleys so close to yet so far from the **CRG Boulevard** sometimes? It could be more confusing from not knowing in what make-up of "knowledge" we are in for. While going after a niche, the range of purported "**CRG knowledge**" upon further <u>normalization</u> might in parts fall into one of these areas of interest:
>
> - **Crane Runway Girders** in general, as in that for generic structural design- and connection detailing- related measures concerning areas mostly into <u>*Practical Structural Engineering*</u>
> - **Unsymmetrical Sections** in general, as in the process of typical engineering design for analytical purpose requiring the calculation of geometric properties for members with unusual profile configuration concerning areas mostly of <u>*Advanced Topics in Strength of Materials*</u>
> - **Crane Runway Girder specifically of Unsymmetrical Sections**, as in their design treatments and the understanding of their behaviors with consideration of the most-effort-consuming process demanding methodical knowledge in areas combining <u>*Structural Engineering and Engineering Mechanics*</u>
> - **Crane Runway Girder in-service Survey and Inspection**, as part of the facility maintenance programs, operation and management functions requiring <u>*On-site Civil Engineering and Structural Engineering*</u> attention
> - **Crane Runway Girder Failure Analysis**, as part of forensic analysis coupled with field investigation requiring knowledge in <u>*Advanced Structural Engineering*</u>
> - **Crane Runway Girder Repair and Upgrade**, mostly as part of the facility maintenance, expansion and/or life extension program requiring <u>*Project Management and Practical Structural Engineering*</u> attention
>
> What itemized in the list basically covers the "**Full Life Time History of Events**" experienced by a typical **CRG** from head to toes. The scope of coverage certainly went beyond normal activities limited under Pure Structural Engineering Analysis-Design alone

Unless in those regards had we been there, seen a lot (if not all) and done quite a bit (if not all,) otherwise quite often and more aptly our "all-around engineering perceptions" over subjects of lesser-explored territories could only be mediocre at best; that means our *all-around-**CRG**-problem-solving skill* could never be solidified if we don't have ample "clinical" experience in taking good care of all sort of ailing structures of all ages and shapes under all conditions.

> *What's the point?* In absence of solid knowledge base, our (shaky) sentiment of being "so sure" on certain technical undertakings could unexpectedly turn dicey into "not so sure" in the next instant, and vice versa. But such unique vertigo would not sink deep into our everyday "engineering feelings" until after we learned plentiful "hard" lessons from "solid" experience, or take it in that:

- *Among current design customs or rules, some may stay as is, some may need modification, some could be discarded and some new ones may be established, provided that we know which is which, for what applications and situations, etc.*

- *There could be more wrongdoings to **unsymmetrical sectioned CRG** from doing it by the same old technique drawn from what's considered less wrong otherwise elsewhere*

Obviously what needed the most is provision of rules and guidelines on proper engineering treatment to ***unsymmetrical sectioned CRG*** in general; that might not happen until the official **Research and Development** (**R&D**) formally recognized the subject structures as a special class of their own, and for which issued official engineering-design guidelines. Yet not until then, then beware; taking hints (wrongly) through a few **Chapters** of this **Article Series** could be risky if followed the wrong way.

In any event, hopefully the information put forward herein could re-prime our mindset to think it through much more thoroughly next time before accepting untested opinions on **CRG** of unsymmetrical section. Once again, it would be a judgment call on the Readers' side whether to go along with what purported in the Series on **Open Sectioned Crane Runway Girders with Arbitrary Profile Geometry**; among some of the views, obvious or obscuring, were on:

- How to approach a **CRG** with unsymmetrical section
- How to analyze for the structural responses to given loads
- How to justify its adequacy

- How to validate the (sometimes obscure) assumptions that were (blindly) trusted in making same assumptions repeatedly, and more importantly

- How to efficiently execute and document the qualification process by removing "*hidden asterisks or mysteries*" and making clear to ourselves as Preparers and to the Design Reviewers per Quality Assurance intents, etc.

It does, in general, take a whole lot more grounds to "seriously" cover all bases while qualifying **CRG** structures against "metal fatigue" when compared to the effort normally taken for most **non-CRG** members or those free from being haunted by metal fatigue. Such a collective "hard feeling" or "engineering unfairness" should be applicable to **CRG** members of all cross sectional shapes, symmetrical sections or otherwise, soon shall we see why

But in qualifying our **CRG** for requirement per <u>non-fatigue-related</u> design criteria, to defend the position in reducing a structural qualification chore of this magnitude into "checking only the material yield limit <u>as if</u> at this instant ignoring the importance of other modes of failure" is obviously very bold and very controversial, intuitively. Therefore on its worth, before incorporating such a bold and controversial rationale into practice, it should deserve more hands-on How, Why, What and What not in plain language than what is out there.

Hereinafter with all due respect to work of other predecessors on the subject, the contents herein, be that found original or proven not original and whether correct or incorrect as expressed as is in the entire Series, are only the Authors' opinions and are strictly intended **FOR INFORMATION ONLY**.

There is absolutely no educational merit in this, Structural Engineering-wise, because nothing appearing in the Series is **academically** new. However, through the Series, some of the Readers may become aware of subtle promotion of different approaches than others in interpreting certain concepts thereby logically leading to different approaches in problem-solving techniques as well

The purpose of this **Chapter 1** Article is intended to convey the bare essentials of introductory materials, ample enough for understanding the basics of **CRG behavior under intended loadings** due to normal Mill operation and nothing more

There is no need to punch in the calculators in the first few Chapters and neither would there be any real problems solved yet. But before long, if it could encourage further technical advancements and awareness from the **R&D** or persuade more extensive coverage on members of unsymmetrical section from the Regulatory Committees of Interest then the Authors' goal would have been achieved.

And by all means, to any topics of interest, if inspired to go beyond the two cents worth from this effort, Readers on their own attempt whether being misled or confused or not, having more questions or not, shouldn't hesitate to consult other official resources for more in-depth treatments.

1.6 Preparing for the Big Mess

While justifying the structural adequacy of any **CRG** member beyond fulfilling basic flexural design requirements, one must address "torsion" and/or "buckling" one way or the other unless, but no guarantee, the girder has been **effectively** supported at close-enough intervals:

- Against lateral (bursting) movement of components under compression and
- Against (uncontrollable) rotation about the longitudinal axis in general

Notice the two very heavy-duty words tucked in as if describing some form of passive aggression in ultimatum: Bursting and uncontrollable

By normal engineering design and hands-on fabrication detailing customs, obviously achieving such lavish provisions would present a much bigger challenge to **CRG** than from handling other types of **non-CRG** structural members especially for there are so many Mill facilities with so many different **CRG** configurations for so many different functional requirements and operation/maintenance constraints to be dealt with − yes, all these design constraints.

Every now and then being aggressively conservative or declaring statements stressing the point of being conservative may be one of the most convenient ways to circumvent or dissociate from taking proper care of torsion and/or buckling. Such course of action may succeed in certain applications provided that no one caught on to request for validation of how it is done. But that tactic if lacking substantiated backup from detailed computation may not work out, both in logical sense and in legal sense. Why? Because of a couple of reasons, if sincerely doing the real thing in proper manner then:

(1) The qualification of **CRG** structures against metal fatigue and (2) the justification of adequacy for unsymmetrical sectioned member under torsion influence are very challenging and could not be exempted so easily by uncalled-for rhetoric alone without winning a fight − against the **numerical kind of mess** we meant

Wondering what challenge? What fight, from where, and by whom?

There were plenty of that, for instance: Some of the aggression or challenge could be seriously legal involving financial gain/lost, or be plain literal involving personal pride. It could come from all fronts, even coming through from within our own team of associates, critics, appraisers or our Peer Reviewers on Quality Assurance grounds, or from those who care and dare to dig deep and to prove numerically over technical details if any harm could have done onto the structure from being willfully ignorant of certain issues, namely, *location of shear center, special load-response effects (torsion) and/or from some other understated design issues, (metal fatigue) etc*.

Then in the courtrooms where most of the engineering disputes were presided over and/or settled:

> Being too naive or too humble on defending the rightful technical position could be "seen" just as "guilty" as being stubbornly old-schooled conservative with no proviso of compelling design logic that supposedly be backed up with hard numbers/figures otherwise all that could be nothing but pretext of much more (reputational or financial) risks/troubles to come

Luckily if we won our (legal) case after all; but based on experiences that "true luck" is won only if all parties could avoid entering the courtrooms to begin with. And lastly **the unavoidable big mess**: It would be a very expensive and very rough journey to even justify for ourselves when it − the legal tangling matter − does happen unless we are well prepared ahead both knowledge-wise and methodology-wise in defending our engineering-design positions.

1.7 How Should We Begin?

To do the job the proper way, in a long haul, although there is no need to memorize all the theories and all the complex derivations behind closed books but at least whoever is serious in this "business" should be better off from understanding the fundamentals in how **CRG** structure behaves under load.

If we were to start from the scratch then, among others, here is the primary listing of **some** of the important subjects of interest for **CRG** engineering:

- **Material yielding**
- **Local buckling**
- **Lateral torsional buckling**
- **Twisting and**
- **Warping**

The list of subjects appears to be in an ascending order in terms of complexity involved in the comprehension and the treatment to these issues. Without inferring too deeply into the technical prerequisites meanwhile as each topic and the spinout subtopics were limited to only a few paragraphs or pages here and there, some trusts that it might be quicker to get up to speed the hard way in grasping the basics through a reverse order of the subject listing before venturing into other issues of interest.

Let us begin from the girder's <u>global</u> object orientation by fixing a right- or left- hand-ruled **XYZ** system to an open-sectioned member, by familiar setup:

> It doesn't matter at this juncture whichever sense is pointing positive or negative for the system; by whichever chosen arrangement, one could have associated (1) the **Z**-axis with the (pre-deformed) longitudinal fibers and (2) the **Y**-axis with the gravitational axis. It follows that the **X**-axis would be parallel with the **CRG** flanges. Finally it doesn't matter whether purposely or by accident, **Y-**axis may or may not coincide with the girder web centerline
>
> <u>*Except noted otherwise, this chosen system would be generalized throughout this Article Series*</u> (and it should be more helpful to confirm the elected system with a simple sketch as we go)

When choosing global coordinate system at one's own convenience for an <u>arbitrary</u> **CRG**, while not defining exclusively for <u>most of symmetrical sections</u> then, **it is quite likely that**:

- The **XY** origin of the user-axes <u>might not</u> coincide with the **elastic centroid** (*Understanding from one of the simplest characteristics: **Elastic centroid** is the **X'Y'** coordinate pair where the elastic flexural bending stress vanishes. But to avoid any undue confusion, it is safer to always identify the entity "centroid" with a modifier word **elastic** or **plastic** in front*)

- Also, the adopted **X-** and **Y**-axis might not be oriented correspondingly along with the **principal axes X'** and/or **Y'** to start on (*By the same token, **principal axes** should have been designated with the word **elastic** or **plastic** in front. However, given the circumstance herein we could omit the **plastic** designation but would limit the practice to always be in reference to **elastic** behavior only*)

If so satisfied the two preceding conditions then, unless the configuration of our chosen **XY** orthogonal set had been confirmed through prior calculations and had already been preset **intentionally** to the cross section of interest otherwise every geometric relationship would had been **arbitrarily** defined, essentially.

No matter how arbitrary the chosen setup had been, but in correspondence to which, the *flexural moment of inertia* about the user-defined **XY** orthogonal axes through ***elastic centroid*** at nodal coordinate pair $\{\mathbf{X_c}, \mathbf{Y_c}\}$ would = $\{\mathbf{I_x}, \mathbf{I_y}\}$ and the *product moment of inertia* would = $\mathbf{I_{xy}}$, which of course vanishes when either or both **X** and **Y** were also the ***elastic principal axes***.

Assuming **X** is parallel with the primary axis of bending; if **all** the applied loads did pass through the **shear center** of the cross section then either the **X**-load or the **Y**-load would have caused only *flexural bending moments* and *flexural shear forces* while the **Z**-load would impose only *axial effect* if its **XY** load resultant did pass through the elastic centroid.

Or else torsional moment and thus torsional effect would be unavoidable

However, either under a perfect loading arrangement or by sheer coincidence, albeit neither **X**-load nor **Y**-load passes through the shear center **individually**, but so long as their line of **loading resultant** does, thereby there would be no **inherent** torsion per external force equilibrium − obviously such perfect setting would almost always exist in theory and rarely happens in reality due to the dynamic nature of Mill operation. But even if the "perfect setting" does take place then perhaps it would last dynamically only for a flashing second at the most, especially for unsymmetrical sectioned **CRG**.

In other words:
*Given that by functionality the loads were applied at nowhere else but at the top of the crane rail, there would be but a very slim or no chance to evade the torsion's magic charms in all **Crane Runway Girders** of any shapes and of any sizes*

What being said is a very disturbing reality for **CRG** to fall into already, and then on top of that for those members having unsymmetrical section in general, as if there is almost no way out of the shadow because traces of torsion would always reside in them permanently, even under their own dead load (theoretically although in negligible measure.) Readers of all ranks and disciplines are encouraged to be skeptical on this, but after all whoever vehemently denies the existence of such reality is quite welcomed to tread into the wonderland of **CRG**

Considering the generic case for open-sectioned members of any cross-sectional geometry, despite being in a "safe" state explicitly through external force equilibrium, but implicit "torsional effects" could still be **induced** from "buckling" or the aftermath of a buckling failure through local or global instability.

Acknowledging this fact, if the structural system were not properly analyzed or not properly designed or not properly detailed or not properly fabricated or not properly installed or not properly supported or not properly maintained to prevent its occurrence then buckling accompanied with "torsional effects" could happen to any structure owing to imperfections, for the worst in which sometimes with little forewarning even though the member was merely subjected to loads of moderate amount applied by nominal means that do not normally set off intentional torsion – but it could

To avoid being led into a "collapsing state" in the event of buckling or post-buckling, a structural member must be sized and shaped not only physically as strong as adequate strength-wise but also must be logically designed in meeting several other obligations, at the least:

- *To maintain the six-degree-of-freedom global force equilibrium between the applied loads and the support reactions, and*

- *To regulate the internal stress-strain compatibility in keeping pace with the imposed global deformations*

Of course, these obligations won't be met unless the measure of every structural consequence in regards to *action, reaction, deflection, rotation, stress, strain, etc.* stays ***in an elastic state***; or else whether if buckling induces torsion or just the other way around, either incidence could seriously challenge the structural stability and integrity

Intentional torsions take place regularly from two popular effects:

- Either by **direct** application of torsional moments **Mt** (the subscript "**t**" stands for "torque" and the rotation is about the **z**-axis) or

- **Indirectly** from **X/Y** load-resultants not passing <u>through</u> but passing <u>by</u> the **shear center** at notable offsets

But other than from those usual causes just described, ever wonder:

<u>Could torsional effects be brought on by **Z**-load alone?</u>

Typical "Structural Responses" − *deflections, bending moments, torsional moments or flexural shear forces, etc.* − were measures of various effects induced by the externally applied loads. Without a doubt there is *delayed timing difference* between the **actively** applied "Loads" and the **passively** induced Responses or Effects

"Active Loads" and "Passive Effects" should not be mixed up in this context. Notice that only the <u>internal</u> torsional "**Effect**" was of interest here, not the torsion "**Load**" of <u>external</u> source because there isn't any ***intentional torsion*** being implied just yet − not directly induced from counteracting the (standalone) **Z**-load

And so if someone would consider only the mandate per <u>external</u> force equilibrium requirement then the answer to our outstanding **Z**-load question is certainly <u>no</u>. But if going behind the scene to see how <u>internal</u> stress conditions were balanced across a cross section only to maintain internal equilibrium then the answer is definitely <u>yes</u>.

Surprised or not by this truth (or fiction) is one thing, but how could it be? Torsion induced by **Z**-load? Interestingly that could be explained its way out through **Warping**.

1.8 What is Warping?

"Warping" is a tricky structural phenomenon above and beyond casual conception or normal appreciation. For stick-like members of regular configuration, there is no need to be caught up in the details of warping if these members were designed by the Book going along with **simple bending rule**.

But tricky as it is, when not fully comprehending what "Warping" means, this particular event had often been confused with "Twisting" and it shouldn't be. It is difficult to describe what it is in a straight shooting manner. But "Warping" always earns a lot of respect from most Engineers especially when running into members of nonstandard profile geometry − *or simply a plain old unsymmetrical section* − or being forced to deal with Warping Constant- related issues in design sessions, or by curiosity, etc.

Anyhow, anyone attempted to illustrate warping phenomenon through the front entrance must deal with hardcore Engineering Mechanics involving math derivation, which would not be a trendy thing to do for the occasion. Yet the subject event could also be understood through the back door via more friendly course that would usher in a **self-serving answer** without any hard numbers or complex equations (but don't be amazed over how warping is explained this way for this is not new.)

In the process of structural qualification based strictly on "stress criteria," there were only a couple of subjects of interest among all:

- *The shear stress in the "local" **XY** plane and*
- *The longitudinal (or fiber) stress perpendicular to the "local" **XY** plane*

  Incidentally, to tell apart from the **fixed** <u>global coordinate system</u> as the structure is under external loads, the orientation of <u>nodal local coordinate system</u> at a specific node is not fixed but **flows** along with the *instantaneous deformed shape* led by the governing *elastic curve* that varies from node to node along the member length

<u>Hereinafter in order to make a direct connection with the **Z**-loads, one should set focus solely on effect from the longitudinal stress for now.</u>

Imagine that the geometry of a given cross section that has four extremity corner nodes: **A**, **B**, **C**, and **D**; and suppose each node falls within its own quadrant such that:

> ***A** is in the first quadrant, **B** is in the second, **C** is in third and **D** is in fourth*
>
> Upon lining up the quadrant boundaries with elected **XY** Cartesian axes, any two adjacent quadrants can be grouped into a "zone" facing one of "compass orientations" such that:
>
> The "West" side would contain nodes **B** and **C**, "South" side would cover **C** and **D**, zone "East" would include nodes **D** and **A**, and finally the "North" would have **A** and **B**
>
> In an overview, it could be chosen purposely that the section's **elastic centroid** coincides with the coordinate system's origin, a focal point at which all four quadrants meet. It is immaterial whether if the origin matches the **shear center** for the ensuing discussion

What comes next is to investigate the general response of the cross section when being applied an **eccentric** **Z**-load "**Pz**" that lands say, in the first quadrant at some <u>radius</u> from the origin. It follows that the projections of the said offset radius onto the **X-** and **Y-**axis were the component eccentricities designated as **ex** and **ey**, respectively.

A universal sign convention for the longitudinal (fiber) stress should be in order, under which tension is positive (+) while compression is negative (–). Next it's time to **qualitatively** look into the stress distribution **logics** based on the <u>three</u> most obvious circumstances arising out of the simple flexural behavior:

> <u>Firstly</u>, the most **direct** effect from **Pz** is the familiar compressive fiber stress with a **conceptual** intensity of **fa** = **Pz** / **A** where **A** = **effective cross sectional area** that Readers should be in confidence as to why the word **effective** was used here. Provided that (1) the section is fully effective and (2) there is no buckling issues emerging, then **fa** would distribute uniformly over the entire section and spread to all the key nodes in symbolic terms as: –**A**, –**B**, –**C** and –**D**
>
> <u>Secondly</u>, the eccentricity **ey**, in the first quadrant, when coupled with **Pz** would produce a bending moment about the **X**-axis, **Mx** (= **Pz** * **ey**) causing compression in the "North" zone and tension in the "South" zone. The bending stress for a typical node located at normal distance **Cy**

from the centroid could be calculated as **Mx** * **Cy** / **Ix** with distribution into the extremities as –**A**, –**B**, +**C** and +**D**

Likewise <u>thirdly</u> then the eccentricity **ex** would couple with **Pz** yielding a bending moment **My** (= **Pz** * **ex**) about the **Y**-axis. The bending stress at normal distance **Cx** from the elastic centroid becomes **My** * **Cx** / **Iy** resulting into –**A**, +**B**, +**C** and –**D** due to compression on the East and tension on the West

After the stress distribution settled in within each local quadrant, it is necessary to evaluate the equilibrium condition at all the **extremities** making sure that everything is balanced "internally." In other words the attention is focusing on the combined effects from all said cases at **A**, **B**, **C** and **D**.

It makes rational sense to begin the balancing process in those (**B**, **C** and **D**) quadrants that were cleared from application of external force **Pz**. Starting from the fourth quadrant at node **D**, one would first add up the two "–" and the one "+" to arrive at one net "–". Then against that, it must be counterbalanced independently as the newly formed "fourth case" with a "+" sign to stay in check for quadrant **D**

With similar tactics after carrying on with that scheme to the third and the second quadrants, it is necessary to apply one "–" at node **C** and one "+" at node **B** in order to preserve local equilibrium. (<u>It definitely helps with a simple sketch</u>)

So far nothing had been done to quadrant **A** yet, but it did register the net gains and losses based on the balanced state (logic) already reached at other nodes, namely: +**D**, –**C** and +**B**, or grossing two plus signs versus one minus sign. To consolidate the disparity simply take out the single "–" from the two "+" signs and that leaves with one extra "+"

This final leftover "+" has to be neutralized with a single "–" and be accounted for at nowhere else but at node **A**. The newly balanced state at all four quadrants would become the concluding <u>fourth</u> condition. It is the symbolic result of internal stress equilibrium combining all the probable effects, direct axial and flexural bending due to **Pz**.

What becomes of the <u>fourth</u> balanced condition is: +**A**, –**B**, +**C** and –**D**

Noticeably the **self-serving answers** to the question regarding "warping phenomenon" now rests in an interesting fact, in which the fiber stress vector always flips sign (from "+" to "–" or vice versa) between any pair of adjacent key nodes. The sequence of nodal pairing may also be observed (1) by the quadrants: whether clockwise **DCBA** or counterclockwise **ABCD**, or (2) be examined by the compass orientations: North, South, East or West.

<u>The warping phenomenon owing to axial load **Pz** is what exhibited from the sign-alternating +**A**–**B**+**C**–**D** pattern under the fourth balanced condition</u>

Within the cross section's confine, the longitudinal stress having such a 2-way sign flip-flopping feature across contiguous pair of distinctive zones is defined as the <u>warping normal stress</u> $\boldsymbol{\sigma_n}$. Warping normal stress exists or congregates not only at or near the section's extremities but they actually disperse over the entire cross section.

The intensity of warping normal stress within each "+" or "–" zone would always decrease linearly from the peak value at all extremities down towards **zero at the shear center**. We shouldn't be confused by a fact that warping normal stress could also be zero at the **elastic centroid**, but only true for doubly symmetrical sections.

Although the in-out/plus-minus stress pattern is seemingly unconventional by the goofy "look" or so unexpected by normal "imagination" but the stress distribution logic does follow the implication per (1) torsion theory and (2) the behavior for members of open section, and therefore one should not be surprised by such happening.

> Now even though the cross section appeared free from any (1) external bending moment and (2) intentional torque, but the resulting force vector from any adjacent pair of these longitudinal stress zones would develop into an internal moment resultant, or a moment pair designated here as **Mzw**. Consequently the cross section is now subjected to two opposite sets of "internal" moment of identical quantity but yet independent to each other
>
> Basically this **Mzw** moment pair existed under the fourth balanced condition is a rather unique characteristic, which is neither a torque moment nor a flexural bending moment but the formation of force resultants from the variation of **Z**-stresses, and it is the so-called **bi-moment**

Interestingly, when similar situation was applied to a member with symmetrical **I**-shaped section, as would be directed by (1) its unique as-given profile geometric feature and (2) the as-calculated innate section properties, the numerical dispersion of the bi-moment pair would compel the effect as if converging into the two flanges, which happens to resemble the **mirage** that:

> Both (flange) elements were deflecting laterally along opposite **X**-directions away from each other with respect to the **Y**-direction and thus (1) as if each flange would experience flexural bending independently about the flange's own strong axis and (2) as if the web is doing nothing
>
> *Isn't this the notorious **Lateral Bending** or **Flexural Analogy**?*

Speaking of warping phenomenon, as it is closely associated with the bi-moment as described earlier, one should never confuse the bi-moment with its first cousin entity "**warping torsion moment**" but read on for more on that later.

Before leaving the subject discussion on "*warping owing to axial force*" we need to summarize what the cross section is ended up with by now since after the "effects" from **Pz** were broken up into four cases representing **four distinctive equilibrium constituents**.

> By tallying up the effect from these four cases together, all the "+" and "–" signs would cancel out at nodes **B**, **C** and **D** leading to perfect balanced state in all respective quadrants, except at node **A** where it racked up four logical counts of "–" sign (check the sketch if there is one drawn)
>
> Recognizing that node **A** is in the first quadrant, and so is applied load –**Pz**. The net sum of internal responses at first quadrant could be **qualitatively** regarded as "–4" longitudinal stress **units**, which bears a fictitious dimensional unit (or intensity) in this example
>
> Presumably in order to arrive at a gross equilibrium condition, the *internal force sum* must be balanced with the *external force sum*; by the way, the actual equilibrium condition should be substantiated by hard numbers. Of all account whether reasoning by theory or by carrying out in real applications, the stress **integral** involving the fictitious "–4" units must equal to the applied load –**Pz.**

What as described up to this point is the detailed explanation of how could warping phenomenon occur as a response mainly due to axial load **Pz** being applied with an offset from the **shear center** – through it all only for purpose of understanding how does a typical **XY** cross sectional plane may "deform" under the influence of warping, and nothing more.

But don't be mistaken that being the only basis that leads to warping occurrences. The fact is, even lacking **Pz**'s influence such as (1) when **Pz** = 0 or (2) when **Pz** ≠ 0 but passing through **shear center**, sure enough warping can always be instigated by intentional torsion **Mt**.

1.9 What Could Bi-Moment Do to the Cross Section?

As already illustrated, it induces warping normal stress $\boldsymbol{\sigma_n}$. The orientation of warping "normal" stress acts into/out of the (local) **XY** plane, which is in parallel with the (local) **Z**-axis, basically, just as any other variety of longitudinal stress does.

Interestingly for open sectioned members, warping normal stress $\boldsymbol{\sigma_n}$ shares some similarity (and dissimilarity) with flexural bending stress $\mathbf{f_b}$ along several lines:

(a) Both brands of stress would peak at the "outstanding extremity" of each and every component element. *In terms of structural modeling of a local profile component element: An extremity is usually the terminal node, i.e. a free node or an unsupported node*

(b) All in all, flexural bending stress decreases from the element's extremity toward the cross section's interior and does vanish at the **elastic centroid**. In similar way so does warping normal stress in trend but vanishes at the **shear center**

(c) Flexural bending stresses $\mathbf{f_b}$ at anywhere located at a *normal distance* "**c**" perpendicular to the **elastic principal axis** − centroidal axis or axis of zero bending stress − could be calculated through principal moment of inertia as $\mathbf{f_b = M * c / I}$ whereas warping normal stresses $\boldsymbol{\sigma_n}$ with respect to the **shear center** could be calculated as $\boldsymbol{\sigma_n} = \mathbf{Mzw} * \boldsymbol{\omega_n} / \mathbf{Cw}$

    Both $\mathbf{f_b}$ and $\boldsymbol{\sigma_n}$ formulas look very much akin to each other in the arrangement of terms in that **Mzw** is the bi-moment, $\boldsymbol{\omega_n}$ is a torsional section property known as Unit Warping, Normalized Unit Warping, or simply the Warping Function, and **Cw** is the notorious Warping Constant

When subjected to **XY** loads under *normal operating* condition, provided that the bending stress $\mathbf{f_b}$ were maintained in an **elastic state** at magnitudes below the yield limit, the deformed plane of the member's cross section would always stay in perpendicular to the local axial (longitudinal) fibers at all times. Thereby along the member's length between supports from one end to the other, the cross section's **neutral axis** would be deflected into a "curve" – or elastic curve – following a uniquely prescribed geometry.

> The global deformation curvature due to flexural bending from **XY** loads could be formulated into a simple algebraic function or a set of functions that describe the governing ***elastic curve***
>
> The "shape of the curve" − deviation from being straight − is highly dependent upon the *applied load configuration, cross section geometric properties, support boundary conditions and the material properties, etc.* And under **elastic conditions** regardless to how may the *local* curvature resemble, *the cross sectional plane at any and all localities would always remain being "Plane" before, during and after the load applications*

On the other hand, the cross section's "Planar" situation could take a "sharp turn" owing to extant warping normal stress $\boldsymbol{\sigma_n}$:

> Under its spell each longitudinal fiber strand – individual thread within any slice of section profile − would either extend or contract except at the **shear center node** as already evident through the +**A**–**B**+**C**–**D** pattern demonstrated in an earlier example. Therefore the cross sectional "Plane" would appear distorted unevenly under the **Mzw** (or $\boldsymbol{\sigma_n}$) influence, or simply stating that the *"Plane" would no longer remain "Plane" anymore*

1.10 What Could Torsion Do To An Open Section?

A better entry point into the generic torsion behavior of opened sectioned members may be from understanding − not solving − the primary unknown term in the governing differential equation; and that unknown term is $\boldsymbol{\theta}$ or $\boldsymbol{\theta z}$, which is the angular rotation about a longitudinal axis drawn in parallel with the

Z-axis passing through the **shear center**. An important pointer: **θ,** in simplification, is precisely the **θz** as stated, which is neither **θx** nor **θy** − the flexural counterpart.

> The solution function of **θ** together with its first three successive derivatives with respect to the **Z**-coordinate, **θ'**, **θ''** and **θ'''**, were the most significant math expressions needed for evaluating torsional load responses of <u>open-sectioned</u> members
>
> Of *engineering analytical-design significance* in the world of torsion as far as structural behavior of **Crane Runway Girder** is concerned, open-sectioned members are exposed to three distinctive categories of torsional moment. Each moment category (non-bending-related) is defined through a function of its own respective order of derivative of **θ**; and in turn each moment category (or the associated math function) is accountable for a unique brand of torsional stress

The classic close-formed solution of **θ** and its derivatives −already worked out for prismatic members having the most-common boundary conditions under various loading patterns − were published in many general Textbooks and Design Guides and References on the subject.

> However if in need of a *systematic treatment* to the mathematical family of **θ**-related functions among others, it could be looked up in one of the most popular references:
>
> "**Roark's Formulas for Stress & Strains**" (**Roark's**) by Warren C. Young
>
> **Roark's** has been recognized as one of the most indispensable "must-have" standard references in the field of Engineering Mechanics. Its coverage on many interesting subjects beyond torsion was notoriously comprehensive and had been appreciated for "about" a century as of this writing
>
> There is no point in repeating the various expressions for **θ** or its derivatives in here. But before making senses of torsion and prior to performing any wide-ranging comparison with the flexural behaviors it is imperative to be familiar with the basic characteristics and what comes about from each function (**θ, θ'**, **θ''** and **θ'''**) in the first place; otherwise it could only get increasingly confusing from here on if not setting apart clearly over what is and what isn't

So what could torsion, or its representatives **θ, θ'**, **θ''** and **θ'''**, do to an opened section?

<u>**First of all**</u>, **θ** is merely a solution function describing the global deformation geometry in terms of angular rotation about the **Z**-axis through **shear center**.

> As a matter of fact, the numerical significance of **θ** is of very "small" quantity when expressed in **Radians**. It doesn't seem to earn much respect from the mainstream on its inherent numerical quietness, or perhaps because it harvests neither any moments nor any stresses <u>directly</u>
>
> *However on the quantity of rotation, it could be a big engineering blunder should anyone (1) calculate wrongly <u>by accident</u> or (2) guesstimate wrongly <u>by choice</u> (using approximation) and/or (3) <u>overlook</u> the impact it had on **CRG** serviceability*

Whenever "deflection" is cited in the design of conventional structures, most Engineers intuitively make connection with the **linear** translations **δx** and **δy** − *the rigid-body movements* − owing to flexure from **X/Y** loads and hardly critic any further than that in terms of ramifications from the **rotational** counterpart, **θz**.

> But if "inexperienced" in handling **CRG** serviceability issues (through appropriate means or not) then, even the "most experienced" among us could be fooled into thinking that it must be **OK** to forsake **θz** since most Textbooks or Design Guides on **CRG** tend to concentrate so heavily on rigid-body movements from **X/Y** loads through **elastic centroid** and <u>seldom</u> or <u>never</u> seriously address the problem with **shear-center-based** "rotation **θz**" per se, so it's as if no harm to articulate the "why bother" phrase in that regard

Yet after all, there may be legitimate or reasonable excuses for maintaining such "why bother" or "no bother" attitudes when designing <u>conventional</u> structures under <u>conventional</u> loads from <u>conventional</u> applications and most notably when justified, so long as the "**effect**" from **θ** is indeed numerically negligible

Then on the other hand:

*Even if we elect to have some "bother" with **θ** but there is basically no **rational** ways to seriously "bother" with it and for that matter, should one have already chosen to fudge the numerical equivalence over accuracy on either the structural responses or the structural behaviors due to torsion while using **Flexural Analogy**, then watch out*

There could be some other opinions given here on what to do. But it is of no point getting into heated discussion over what "attitude" or "analogy" should be or shouldn't be taken towards certain engineering particulars, conventional or non-conventional. But unquestionably that **CRG**, being recognized more so as a non-conventional structural entity than being the opposite, should be treated with extra care with respect to the deformation not only in the direct "**effect**" from **θ** but also in all of its derived effects as well

While following proper engineering etiquette in proviso for serviceability's sake, it is always better in all situations to calculate the "more accurate" value of **θ** proving the case by realistic numbers rather than evading from doing the chore using approximation or that with a "why bother" stance. Besides that we should ask ourselves:

*Without an accurate **θ** function defined in the first place, how can **θ', θ'', θ'''** and the other torsion-related entities be any (more) accurate*?

If we were fully prepared and intended to be all-inclusive **Quality Assurance**-**Compliant** in terms of collecting the genuine structural response data per se then, it is important to consider the implication from those respective "tangential components" being projected along both **X-** and **Y-** axes attributed to the rotation **θ**, even though in reality the numerical significance of which (in **Radians**) was commonly trailing more or less behind a string of zeroes at several places after the decimal point.

Just follow along with a couple of points here:

(1) Imagine at a given node of interest in the **2-D** cross sectional plane located at a certain **Z**-coordinate, and (2) if letting $\mathbf{r_y}$ and $\mathbf{r_x}$ be the normal nodal distance (projection) from the **shear center**, then the said tangential components could be calculated as $\mathbf{\theta_z * r_y}$ and $\mathbf{\theta_z * r_x}$, respectively − makes sense, but what's the big deal?

Obviously if only proven by calculations, some of the experienced engineers could readily testify that although **θ** might ≈ 0 but $\mathbf{\theta * r_y}$ and $\mathbf{\theta * r_x}$ were nothing but – think about the impact, some of the $\mathbf{r_y}$ and $\mathbf{r_x}$ could be 30", 40" or more. Nevertheless to be considered anything meaningful, these tangential components are additive to the respective rigid-body deflection **δx** and **δy** per flexural effect in arriving at the final total deflection vectors

Once again in order to take better care of the **CRG** serviceability matter properly, the displacements at any **3D** "key" node(s) or key location(s) must be carefully reviewed based on **realistic calculation** with hard numbers from the combination of linear flexural deflection and tangential component per rotation, and definitely not by ***Flexural Analogy***

Consider **CRG** deformation (linear and/or rotational) through any cross sections, one of the most important key nodes **for practical purposes** should be at the **top of the crane rail**. In order to better protect both the crane assembly and the rails it runs on, Structural Engineers must honor the limits on how much deformation could be tolerated at the crane railhead elevation and assess accordingly as first priority more than anywhere else

On more dynamic situations as crane is in motion, to better protect the service crewmembers working near or right next to the runway girder components, say, hot rail or jacking beam, etc., we, Structural Engineers, must minimize the deflection sums, ($\delta x + \theta * r_y$) and ($\delta y + \theta * r_x$) at locations where some of the (temporary) attachments were sensitive to excessive movement, or at the fringe of the "walkway plate" if it were physically associated with (or connected to) the girder

As simple proposition on serviceability's behalf:

If we were given a girder shaped relatively "deep" and "narrow" or "lopsided" then, we should always bestow serious "bother" to **θ**; or in other words, the value of **θ** should be realistically calculated and its effects should be evaluated for girders of all sizes and shapes.

The first derivative θ' relates directly to "**pure torsion moment** or **St. Venant pure torsion**" M_{z0}. Whenever a structural member is under external torque **Mt** then M_{z0} becomes the most common form of torsional response moment that exists in cross section of all shapes and features, be their profile opened or closed, symmetrical or unsymmetrical.

M_{z0} induces pure torsional stress or **St. Venant** pure torsional stress τ_0. This particular brand of shear stress is reversible in all **CRG** applications. It stays in the local **XY** plane and varies linearly across the element's thickness. The distribution of τ_0 could be drawn up as if piercing into the element through the element thickness in a slash pursuing a triangular or slant pattern from the positive (inner/outer) surface to the negative (outer/inner) surface, or vice versa, with a maximum value occurring at the element surface that reduces linearly towards a zero value at the mid-thickness.

The maximum magnitude of τ_0 stress although changes from one element butting another interfacing element(s) but retains the peak-value feature when traversing along the periphery boundary of the member cross-section as if chasing after an uninterrupted loop.

The second derivative θ'' is directly proportional to the "**bi-moment**" **Mzw**. It induces reversible warping normal stress σ_n parallel with the **z**-axis, which varies throughout the entire cross section in its own special stress pattern as already covered previously.

The third derivative θ''' would relate to "**warping torsion moment**" **Mzt**, which induces reversible warping shear stress **τ** in the **XY**-plane that remains constant through the element thickness. The distribution **pattern** of **τ** is very similar to that of the flexural shear stresses or the so-called horizontal shear stresses, the flexure counterpart.

1.11 How Similar Is Torsion to Flexure?

Bringing Torsion and Flexure together into engineering design consideration is no surprise. But looking for similarity may seem little strange for these two fellows were far apart on their face values.

Yes indeed, from a load-application point of view, they are just two independent entities; but if the question were phrased as "**How Different is Torsion From Flexure**?" then by further examination we may find that they also share dire similarity in the formulation for various structural responses and their by-products, one of which has already transpired from the foregoing **M * C / I** and **Mzw** * ω_n / **Cw** expressions. Beyond that example among others, as would be demonstrated hereon, are contrasts in other areas that may be of interest if only scratching the surface a little deeper

Through various venues the similarities between flexure and torsion could be brought out from the settings of several interesting formulas as the subject continues on.

Starting out with instance from elementary flexural behavior of a stick-like member under **Y**-load alone:

If letting **y** = vertical displacement along one of the principal axes then its 1st derivative **y' = dy / dz** would represent the slope of the elastic curve due to that load

By defining an **E I** bundle that stands for the flexure rigidity:

The 2nd derivative of the elastic curve **y'' = d^2y / dz^2** becomes the curvature with an approximation **≈ M / E I**, thereby one could substitute the bending moment by equating **M = E I y''.** Derived from the moment **M** would be the flexural bending stress f_b. Finally the 3rd derivative **y'''** when multiplied by **E I** would become the flexural shear force **V = E I y'''**

It was just refreshed from a branded relationship after Strength of Materials:

The flexural shear **V** is the "rate of change" in bending moment **M**. In simple technical terms: Flexural shear is calculated as the derivative of bending moment, **V = dM / dz**, or simply interpreted which as the "tangent or slope" of the bending moment diagram

When mimicking the foregone expression **V = dM / dz** in torsion theory:

One recognizes a very similar relationship: **Mzt = d (Mzw) / dz**. Herein **Mzt** is the familiar "**warping torsion moment**" based on the third derivative **θ'''**. Or the expression could be interpreted as: The warping torsion moment is the rate of change in "bi-moment **Mzw**." *Through this formula it officially ties in the relationship between bi-moment and warping torsion moment but via a different route*

In Engineering Mechanics the "angular rotation **θ** due to torsion" should deserve the same recognition as the "linear displacement **y** in flexure" has. But only when putting down both "**y**" and "**θ**" matters on the same level then it becomes much more evident as we seriously decode the difference or the similarity between each other's 2nd and 3rd derivatives

In the world of flexure, **y''** relates to bending moment **M** and longitudinal bending stress f_b while **y'''** relates to flexural shear **V** and shear stress **fv**. In the world of torsion, **θ''** relates to bi-moment **Mzw** and warping normal stress σ_n while **θ'''** relates to warping torsion moment **Mzt** and warping shear stress **τ**. In terms of name-calling on different varieties of stress between flexure and torsion, one could hardly ignore the subtle emphasis here linking "longitudinal" to "normal" and "shear" to "shear" when pairing flexure with torsion through literal association

Now that after understanding how **V** relates to flexural shear force and how **Mzt** is to warping torsion moment, why bringing them together here for?

To exploit **V** more expressively, one needs to be brought back to its flexural relevance:

The flexural shear stress (or the so-called horizontal shear stress) f_v could be calculated by the familiar formula f_v = **V * Q** / (**I t**) where **t** = element thickness, **I** = principal axis based moment of inertia and **Q** = principal axis based static moment about the corresponding principal axis through elastic **centroid**

Matching the similarity to f_v = **V * Q** / (**I t**) term by term were one of the torsion formulations in that **τ = Mzt * Sw** / (**Cw t**) where **Sw** = **Warping Static Moment, Cw** = warping constant and **τ** is the **warping shear stress**

When all is said and done, the warping normal stress σ_n could be obtained immediately from the expression **Mzw** * ω_n / **Cw** once **θ''** is known. But because **Mzt** is a derivative from **Mzw**, or as a triple-primed function **θ'''** so that is the reason why warping shear stress cannot be proportioned directly from **Mzw** using simple algebraic manipulation and vice versa, even though they may be related by performing numerical differentiation and/or derivative from/into each other.

1.12 What Is Twisting?

Not very far back from a scenario in that, it was concluded on how warping could be induced implicitly from internal stress equilibrium condition owing to an eccentrically applied axial load **Pz** acting at a certain distance or radius away from the **elastic centroid/shear center**. Noticing in that the equilibrium setup for the **Pz** influence does not directly implicate any "**twisting**" behavior or phenomenon at all.

By twisting we mean a general profile "remaining in its local **XY** plane" being rotated about a particular longitudinal axis that passes through the **shear center**. But the question is:

> *What could happen if a direct external torque (about an axis parallel to the **Z**-axis designated **Mt**) were applied (or resolved) to an open-sectioned member? − Twisting and warping*

When an open-sectioned member is subjected to an external torque **Mt**, the formulation of angular rotation function **θ** (or **θz**) of the cross section about a longitudinal axis passing through the **shear center** automatically becomes the primary focal point. In plain language for torsion's sake, one would like to know what **θ** is and how may it vary from one support end to the other support end.

There were several approaches in establishing the governing equation for torsion, but it is not the goal to do the real thing here. Only to help better understanding the structural behavior due to **Mt**, one could always simplify the theory into a much friendlier edition on torsional responses conceptually, whether deriving it forward from *force (torque) equilibrium* or backward from *displacement (rotation) compatibility* and it would lead to the same formulation.

By organizing:

- The unknown angle of twist **θ**
- The unknown linear translations **x** and **y**, and
- The inferences from their derivatives (internal shear, moments, etc.) at equilibrium

The internal *deflection-rotation* model and the *force-moment* model could be dissected in much greater details using simple algebraic terms established through **relationships** based on:

- The section **geometry** and
- The **elasticity** of the material

It may not be necessary but good to know that among the participating parameters in supporting such relationships were *polar angular rotation, curvature, tangential displacement, Young's modulus, shear modulus, the coordinates of principal **elastic centroid** and most importantly the **shear center**, etc.*

The result came out of the preceding process would be the basic elastic torsional stress-strain relationship. What could be realized from that were the two main varieties of torsional response:

(a) The **St. Venant** pure torsional stress $\mathbf{v_0}$ representing the "**uniform** torsion" behavior in that the sectional plane remains *plane* and

(b) The warping shear stress $\boldsymbol{\tau}$ representing the "**non-uniform** torsion" or "warping" behavior in that the sectional plane becomes *warped*

The problem at this stage is that both the **St. Venant** pure torsional stress $\mathbf{v_0}$ and the warping shear stress $\boldsymbol{\tau}$ are of unknown quantities. Somehow one should take advantages of the facts that:

(a) The value of $\mathbf{v_0}$ is in **direct** proportion to the element thickness but an **inverse** proportion to the torsional constant **J** and

(b) The value of $\boldsymbol{\tau}$ is proportional to warping constant **Cw**

To eliminate some of these unknowns, the logical treatment would be:

(a) To perform numerical integration to the assortment of stresses over the full cross sectional area and
(b) To relate the results to some familiar (non-gibberish) terms

From the foregoing procedure one would end up with two distinctive forms of torsional moment. The **St. Venant** pure torsional stress would lead to **St. Venant** pure torsion, $\mathbf{Mz_0}$ while the warping shear stress would lead to warping torsion moment, **Mzt**.

The definition for both $\mathbf{M_{z0}}$ and **Mzt** had been briefly treated beforehand, and notice that interestingly herein the bi-moment **Mzw** is not even in the picture at all. Readers are encouraged to give their own answers as to why it is so. But anyhow one could finally link up to the total cross-sectional torque or total cross-sectional torsional moment **Mz** with two simple forms of internal torsional moment:

(a) Pure torsion $\mathbf{M_{z0}}$, and
(b) Warping torsion moment **Mzt**

The final differential equation that satisfies the static equilibrium condition expressed in symbolic form now becomes:

$$\mathbf{Mz = M_{z0} + Mzt}$$

And the equation is ready for solution, on which Readers could dip into the details found elsewhere. But let us pause for a few questions of interest:

- First of all, what happened to **Mt**?

  Mt is still sitting there intact, but one should make a note that it is an externally applied torque. Its participation into the solution of $\boldsymbol{\theta}$ is through two other mathematical terms: (1) that directly involves **Mt** and (2) that comes indirectly through the reaction end torque **Mze** that **Mt** had caused at the support nodes

  What should catch one's attention the most is the fact that the end-torque **Mze** is only applicable at the support

  > Although conceptually it is similar to the idea of Fixed-End-Moment in flexural sense for boundary condition of being flexurally fixed – either at one end or at both ends − except that in torsion sense herein **Mze** would always subsist under all end conditions *even when both ends were torsionally simply supported*
  >
  > Anyhow **Mze** could always be derived from **Mt** based on given *torsional boundary conditions, material/section properties and* ***simple*** *static equilibrium*. By the way, but rather sorry to say that the last statement is only half true: It is indeed a **simple** case if only the boundary conditions at both ends were torsionally simply supported, otherwise it could be **extremely difficult** to formulate/calculate the value of **Mze** whenever either end is fixed or both ends are fixed

  Difficulties in handling fixed-end **Mze** could be the major reason if not the sole reason why most of the examples found in most Textbooks on design of open-sectioned steel members subject to torsion were almost always geared for simply supported conditions at both ends. Readers are encouraged to experience their hands on cases involving end conditions other then that. Say, fixed at one end and simply supported at the other end and see how messy it could get. Or a quick peek into **Roark's** to appreciate how complex it is

- Aren't **Mt** and **Mz** the same thing?

  No, never will they be. **Mt** is merely an externally applied vector and is very straightforward in its definition while **Mz** is an internal torque varying from one node to next node, and it could look quite unpleasant once it came out from behind the closed doors (books)

  In fact **Mz** is a function involving several parameters, among them: (1) **Mze**, (2) **Mt**, (3) the location where **Mt** is applied, (4) member length and (5) the **z**-coordinate of the cross section of interest

  Technically **Mz** is a variable along the member length while **Mt** remains being constant and at any rate, **Mt** and **Mz** are not the same

- Why isn't **Mt** in the equation?

  The equation $\mathbf{Mz} = \mathbf{M_{z0}} + \mathbf{Mzt}$ deals with equilibrium of **internal** torsions; **Mt** is excluded from internal matters because it is an **external** torque

- Finally, isn't **Mzt** the lateral bending?

  Mzt has been "interpreted" as the lateral bending in many Textbooks stemming from **Flexural Analogy**. The interpretation, when chosen, may be less critical in applications involving only symmetrical sections (if not more or less appropriate) and perhaps with mindset having no concern of being inaccurate in the amount of deflection by disregarding the warping effect

  Yet Flexural Analogy appears "validated" only on the mirage by a similarity in the "look" of the lateral deflection of flanges compared to the lateral **x**-component (projection) of angular rotation due to torsion

  But herein for unsymmetrical sections' sake, lateral bending may be a hard sell in engineering principle in some (or all) cases; because for these sections, the **shear center** could sometimes be located at a nodal point way out of the section's profile geometric confine. Therefore unless someone insists on reaping the benefit (or the risk) from using **flexural Analogy** otherwise **Mzt** should always be recognized as nothing else but warping torsion moment

Anyhow there could be more questions of special interest needing more answers but there seemed quite enough discourse for our purpose already.

Basically, **twisting** is the general structural response due to pure torsion $\mathbf{M_{z0}}$. The torsional shear stress $\boldsymbol{\tau_0}$ owing to this unique effect could be computed from the formula $\boldsymbol{\tau_0} = \mathbf{M_{z0}} * \mathbf{t} / \mathbf{J}$. Doesn't this, again, look like **M c** / **I**?

Under the influence from pure torsion, the cross section could be portrayed as being "twisted," "tilted" or "rotated" yet being confined only in the **X-Y** plane about the **shear center** *without stretching or contracting* any of the longitudinal **Z** fibers. Therefore the sectional "Plane" would remain being "Plane" prior to, during and after being deformed under pure torsion. And from the cross section's deformation point of view this is one of the major differences between twisting and warping.

1.13 What Makes CRG Structures Stand Out?

CRG application calls for serious engineering dealing with the combined effects from both flexure and torsion for a fact.

Through "**What is Warping,**" "**What is Twisting**" and "**How Similar is Torsion to Flexure**" we should make out a much sharper image into (1) why and how **CRG** would inevitably experience both flexural and torsional effects and (2) how it may behave as natural response to <u>loads applied from the top of the rail</u> at offset from the **shear center**. Collectively all that would frame up a unique design condition complex enough in making **CRG** stand out from most other classes of structure.

Taking into account the combined influence from both flexure and torsion in a typical **CRG** design session in "proper manner" for sure should cause more severe engineering annoyance than relying on **Flexural Analogy**. *But what makes **CRG** structure suffer a notch above most other **non-CRG** structures could have suffered is that **CRG** has to sustain the relentless attack from stress reversals/fluctuation that could cause metal fatigue if not fittingly designed against its occurrence*.

We should be confident by now that not only making good on those technical nuisances as said is a tedious chore already but also the involvement of those <u>**CRG** with unsymmetrical section</u> should stick out more than a sore thumb to all the Structural Engineers for another reason:

> *Pin-pointing the exact whereabouts of **shear center** could be the ever-illusive and indispensable focal point of all things torsion*

Merely understanding the fact that structural behavior due to flexure is <u>elastic centroid-based</u> whereas that under torsion is <u>shear center-based</u> is barely enough. Beyond our normal train of thought, here are several compelling points, which should be qualified enough as better reasons why we should call for further attention to the offshoots from such structural behavior:

- Recognizing accordingly that: (1) Out of all categories of internal nodal stress, although some could be sharing common vector sense(s) but the source of stress could be elastic centroid-based and be shear center-based as well, and (2) each unique brand of stress follows its own unique dispersion pattern across the section profile with distinctive peaks and valleys, which are not likely to share with other brands of stress

- *The <u>calculated</u> coordinates whether identifying the **elastic centroid** or the **shear center** must be accurate otherwise beware of garbage-in-garbage-out situation*

- *There is no easy means to decipher if the stress at any specific node of a specific brand would experience fluctuation and/or reversal effect beyond fatigue allowable(s)*

- *There is no easy means to differentiate directly from looking at the cross section geometry if the torsional behavior of our structure is (1) pure torsion dominant, or (2) warping torsion dominant or (3) mixed torsion dominant unless we drill the number properly*

- *There is no easy means to discriminate immediately from barely the analytical results if the final design is to be non-fatigue-dominant or fatigue-dominant, again unless we drill the number properly*

Enough said, but generically there are only two kinds of stress of interest: Longitudinal (fiber) stress and shear stress. More prominently for all structures including **CRG** that (1) flexure would induce both fiber stress and shear stress under its own brand name and (2) likewise torsion would also induce both fiber stress and shear stress of different brand and at different dispersion gradient.

What if sauntering further from knowing what we had picked up so far:

- On **AISC Chapter F**, the provisions intended for "Design of Members for Flexure," would it be more descriptive if added a word **Flexural** to it or modify the titled to "**Design of Members for Flexural Bending**" knowing that flexure implies both bending and shearing that needed be segregated?

- And then for similar reason on **Chapter G** would it be clearer if it were titled as "**Design of Members for Flexural Shear**" to tell it apart from the shear stress induced by torsion?

- And how about **Chapter H**?

1.14 What Is Lateral Torsional Buckling (LTB)?

Among various structural behaviors under flexural influence, **LTB** should rank near the top as one of the most "interesting" subjects in Engineering Mechanics and it should top the list herein as well. But no matter which path is chosen, it is always tricky to recite **LTB** phenomenon in laymen terms and to also avoid stumbling in with several other sound-alike topics (*flexural buckling, flexural torsional buckling and torsional buckling, etc.*)

From many Textbooks and Design Guides on the subject, Structural Engineers were taught to understand **LTB** behaviors based on these key assumptions:

(a) The member section geometry must be <u>symmetrical</u> and
(b) The section deformations, translations and rotation, must be <u>small</u>

It is with these same two key words **symmetrical** and **small** in thoughts, as followed is a simple (or rather lengthy) account of **LTB**.

Starting from a **symmetrical** cross section of multi-element construct subject to strong axis bending moment **Mx** about its major (**X**) axis, a portion of the profile would be in tension while the remainders would be in compression.

> **Ideally** if all is "perfect" and material used is "flawless" then the stress intensity through the cross section should be closely related with innate profile geometry free of irregular patterns
>
> > Also true per elastic theory, the longitudinal stress flux in the fibers within any typical element would possess smooth patterns either uniformly across or be at <u>linear gradient</u> either in parallel with or orthogonal to the selected axis of reference, varying in rates depending on the axis' orientation
>
> What is not so ideal but inevitable in the real world is **imperfection**
>
> > It exists in almost all structures. Imperfections may emerge or intertwine from defects as natural aspect of material, geometry, loading, fabrication and/or construction, etc. One of the ill effects from imperfection is causing "**irregularity**" or "**disturbance**" in the fiber stress distribution. − it is a separate issue at this point whether the rate of "irregularity or deviation from the norm" is within or has exceeded the practical tolerances mandated under various criteria
> >
> > As a result, the stress distribution within the **disturbed** zone or element(s) would no longer be in uniform gradient (or per nodal-coordinate-wise linearly proportional to the reference axes.) This non-linear/non-uniform pattern could be the results from extensive stress concentration or local spikes. Collectively, the disturbance in stress distribution (pattern) always does more harm to elements in compression than to those in tension

> But normally "minor disturbance" would post no imminent danger to depose element stability until the bending moment **Mx** about the strong axis reaches a critical value when coupled that with inadequate or a lack of support for the <u>critical element in compression</u>

Within the critical element's confine, once the intensity of stress distribution became non-linear/non-uniform along its depth, length or breadth, a "lost of symmetry" about the innate axis-of-symmetry (**IAOS**) would take shape. It leads to an unbalanced state of stress or strain, in which:

(a) The stress intensity would remain in a constant slope on one side of the **IAOS**

(b) But the stress flux on the opposite divide of the **IAOS** could have been settled in uniformly or linearly, except for some localized "extra squeezing" due to the alleged imbalance

As **Mx** kept increasing up to a threshold value, those local fibers already of irregular patterns could be squeezed further that would enhance the formation of irregular stress peaks and valleys that locally the element could no longer accommodate the imposed demand in storing up excessive compressive strain, things happen:

> Needing extra roominess for shrinking/expanding in response to being compressed further but given no instantaneous spare, the offending fibers would have to bring about a sudden burst off (sidestepping) as natural way of relieving of strain energy into whichever the weakest (lateral) direction there is
>
> This incidence would lead to the onset of element's instability or lateral buckling; meanwhile it draws the entire element, which is in (whether full or partial) compression, into that same direction, along which the "gross section" offers the least resistance in that course

Since the originating flexural moment of interest **Mx** is bending about the **X** principal axis, it can be demonstrated (by simple math) that the moment of inertia summed about axes of any other orientation other than the **X** principal axis is always less than the principal major value i.e. **Ix**. Therefore the sudden burst would have to initiate in parallel to the weakest **X** orientation (moving away from the **Y**-axis) that matches up to the least (principal minor) moment of inertia **Iy**. In engineering mechanics, this sudden burst is **Lateral Torsional Buckling** in action attributed from bending about the principal major axis.

Conversely **LTB** could never occur to an open section subjected to load resultants into bending about the weaker principal plane (or from bending about the principal minor **Y**-axis) because the moment of inertia associated with any other axes of bending is always greater (or stiffer) than the least amount of moment of inertia **Iy**.

Prior to bringing on **LTB**, chances are that any slightest lateral movement of a component could end up being the culprit toppling the steadiness of an entire beam − or by luck such toppling may never realize. But there is more to that if it does; any unsteadiness triggered from a local beam (or beam-column) could in turn spread **unexpectedly** to the entire global system, setting off devastating consequences in chain reactions **if** the <u>system as a gross entity</u> were not properly or adequately braced, supported or anchored.

Associating **LTB** event with "material imperfection" or "disturbance in stress distribution" merely makes the point across for the sake of understanding what might cause such behavior. In fact, imperfections could be blamed as anything out of norm and were mostly **intangible**. However, it would be counterproductive if someone would feverishly look for "imperfections" or find sensible ways dodging or mitigating them and then fail to recognize that **LTB** would always occur whenever <u>**Mx** reaches or exceeds a critical value</u> for those girders with underprivileged attributes no matter what caused it.

Besides "imperfections" or even without their presence, the probability of **LTB** occurrence for open-sectioned members may escalate in direct/indirect proportion to several other **tangible** parameters. Among them: *Member's unsupported length, bending rigidity, torsional rigidity, depth-to-width ratio or the* ***Ix / Iy*** *ratio, etc.* For those reasons, optimizing some of these parameters in the design should be a better bet than by other means against **LTB** incidences.

When the prescribed attributes of an open-sectioned beam were in favor of inflicting **LTB**, it could go off owing to loads applied in moderate quantity, even though the initial line of load action is perfectly symmetrical about all the relevant axes of importance. That explains why **LTB** tends to strike more frequently <u>during erection phases</u>, in particular when the <u>intended</u> lateral supports were not yet in place or fully secured, or simply from the mishandling.

> Long slim members such as open-web bar joists should be the most disadvantaged among all **non-CRG** structures. They are prone to classic **LTB** during construction phase most often from lack of proper (temporary or permanent) bridging or shoring. Therefore if given not much of choices and had to live with certain unfavorable attributes but would like to minimize **LTB** occurrences then for any members, it is imperative to provide adequate support(s) to compression elements at <u>all loading stages</u>, ideally

And then when **LTB** fires up, the offending compression element acts alone by itself; yet in global perspective it simply could not bend or buckle as freely and independently at will as a truly "globally buckled column" did, because the offending element is restrained by the girder web after all.

As **LTB** scenario continues on its course, although the buckled compression element had "guided" an initial movement into one direction, but by pivoting through the transitional web (where **shear center** resides,) the elements in the tension zone would engage into leaning back the opposite way, as if to neutralize a local disorder. On one hand the tensile pullback seems to curtail the net amount of "sway" parallel to the **X**-axis, on the other it incites a "**tilt**" about the **Z**-axis through the **shear center**.

The "unknown" quantity of "**tilt**" leads to an intrinsic torsion **Mz**, which is also of "unknown" quantity. But only through both of these unknown entities, "**tilt**" and **Mz**, that the section would be able to maintain its static equilibrium state. Mathematically the trouble is how to associate one unknown to another unknown of different nature by simple but rational means.

*Under elastic behavior, the linear deformations consequential to flexure were based on "**small** linear displacement" while the rotation due to torsion were based on **small** angle of twist.* By taking advantage of those "**smalls**" it becomes much easier to relegate the relationship between **Mx** and **Mz** through algebraic or geometric association. With simple Sine or Cosine function, the unknown torsional **Mz** can then be expressed as a **vectorial** component from the **global** flexural moment **Mx** projected onto the **local Z**-axis. After doing so, the "exact" quantity of the unknown "tilt" disappears and is no longer an issue.

Prior to being eliminated to arrive at the governing **FTB** differential equation, **Mz** would only serve as a transition term bridging a link between the *in-plane behavior* and the *out-of-plane behavior*. Thereby the **instantaneous state of displacement** of the deformed section would consist of a mixture of:

(a) <u>In-plane translation</u> with one component along **Y** due to flexure bending and another along **X** due to lateral buckling, and

(b) <u>Tilting</u> about a longitudinal axis parallel to **Z**

"Tilting" as a result of relative movement of opposite flanges implies "rotation" about the **shear center**. Its hindsight is the combination of "**twisting**" and "**warping**" of an open section.

> **LTB** is one of the favorite events during which "buckling" and "torsion" become inseparable. That is also the reason why the **LTB** formulation entails both twisting- and warping- related properties: shear modulus (**G**,) **St. Venant** torsion constant (**J**,) **Young's** modulus (**E**) and warping constant ($\mathbf{C_w}$) were in it

Interestingly the close-formed solution to the governing **LTB** equation is actually the Square Root of the Sum of Squares (**SRSS**) of two terms of obvious interest: One of them relates to "**twisting**" that also couples with "**lateral bending**" (or the weak axis bending) while the other term relates to "**warping**" as expected. Their presence is evident in various equations catering **LTB** per **AISC** and many other International Codes; except for the fact that some of the parameter clusters appearing in those equation(s) had been masked heavily in various formats and styles of simplification, substitution or normalization, but all would lead to same result for practical purpose.

1.15 What Is Flexural Torsional Buckling?

<u>Flexural Torsional Buckling</u> sounds like a sibling to <u>Lateral Torsional Buckling</u> with a different first name.

> From a *symmetrical* sections' viewpoint, **LTB** is instigated exclusively from a moment **Mx** bending about the strong axis due to **Y**-load through the web that leads to lateral buckling of the compression flange
>
> However the general term "Lateral buckling" sometimes as simplified in place of "Lateral Torsional Buckling" should not be confused with the classic **Euler** phenomenon of Flexural Buckling (**FB**) which also contains the word "buckling" of course

FB is the consequence of **axial Z**-load with its resultant passing through the cross section's **elastic centroid** with these features:

(a) **Z**-load imposes **uniform** compressive stress (contraction) in each and every longitudinal fiber;

(b) As the magnitude of **Z**-load reaches critical limit, it initiates buckling of the member **without any Z-rotation** − sidestepping into the weakest axis that associates with highest slenderness ratio (the quotient of effective length to radius of gyration or **k L** / **r**)

For members of *symmetrical* section, the technical difference between **LTB** and **FB** lies in (1) the unknown terms to be solved for and (2) the source formulation of respective differential equation.

> The unknown term in **LTB** equation is the critical **Mx** moment (or **Mcr** about the strong **X**-axis) while the unknown variable in **FB** equation is the critical **Z**-load, or inducted from which the uniform stress **Fcr**
>
> In fact, one could probe further by recognizing **FB** as simplified version of the generalized buckling phenomenon with an official term: **Flexural Torsional Buckling** (**FTB**). A generalized **FTB** is applicable to all section shape geometries: Symmetrical or unsymmetrical

Whenever a linear beam (column or beam-column) buckles due to axial **Z**-stress or **Z**-load (not from **Mx**) under **FTB**, its **elastic centroid** would burst away from the axis correlated with the weakest slenderness. With the burst off of the cross section's **elastic centroid**, **shear center** would go along with and be relocated in the **3D** space accordingly.

> The resulting state of displacement departing from the original geometric reference is very similar to that due to **LTB** that is (once again) a combination of translations (along both **X** and **Y**) plus the rotation about a longitudinal **Z**-axis through the **shear center**
>
> To maintain the state of equilibrium under **FTB** for unsymmetrical-sectioned members, it requires three simultaneous differential equations to characterize the 3-Dimensinal stress-strain relationships at buckling

Each equation would represent a discrete stability condition with each catering to a specific component **X**- or **Y**-translation, or **Z**-rotation. The pertinent geometric **interactions** between the **elastic centroid** and **shear center** were fully incorporated into each of the three equations

On account of complexity involved in the Eigenvalues solution process, the critical axial load **Fe** (or **Pcr**) for **FTB** must be solved for as the "lowest real root" from a cubic algebraic equation. An expression already in its normalized form for practical use was given as **AISC Eq**. (**E4-6**) as of this writing, which also takes into account the influence from unbraced length.

In applications when relevance to **shear center** coordinate(s) offset from the **elastic centroid** vanishes mathematically for members of (singly- or doubly-) symmetrical sections, the numerical expression(s) deduced from the generic **FTB** equation becomes much simpler.

After the **AISC** equation **E4-6** is inferred into either **E4-3** or **E4-4** by stripping out one or all term(s) carrying shear center offset coordinate(s) the "**Flexural Torsional Buckling - FTB**" event turns into "**Flexural Buckling - FB**" thanks to the simplification in math expressions along with the literal omission of reference to the remark of torsion.

1.16 What Is Torsional Buckling?

This is another variety of structural instability known as **Pure Torsional Buckling** or simply **Torsional Buckling**. In the simplest form it is a twisting failure in structural members attributed *neither to flexural bending moment nor to torque but to pure axial load without bending*. The solution term for Torsional Buckling (**TB**) could be expressed as critical force (**Fe**) or as critical compressive stress $\boldsymbol{\sigma_{cr}}$ rather than (**Mx**) bending moment.

In order for the axial load as applied to maintain its straightness from one supporting end to the other supporting end through longitudinal fibers without bending, **TB** would presumably take place more often to member profiles of unusual section geometry involving relatively wider-spread of flanges and/or members of very short length. However **TB** can also occur by incorporating other modes of failure to members with very low warping rigidity of relative long length. Typically low warping rigidity is fairly common for sections with component element geometry (as if) radiating from a center node such as cruciforms, angles, double angles and tees, etc.

Judging from a broader perspective, "pure" **TB** alone is not likely to govern the design of **CRG**. But that does not guarantee any relief from infringement by other modes of failure especially due to unusual section geometry and loading nature, from which many forms of imperfections, **TB** and/or local buckling could lead to a generalized **FTB** if not **LTB**.

1.17 What Is the Latest Code Positions on Unsymmetrical Sections?

Let's start from **L**ateral **T**orsional **B**uckling (**LTB**) first. **LTB** is a complex subject made simple as from many sources; Textbooks may differ from one to another in explaining the "why" and the "how" but all would lead to basically the same "what" conclusion, numerically. Vast amount of **R&D** results on **LTB** had already been incorporated into the modern Code objectives suitable for practical uses. But the affluence of information is only helpful mostly, or exclusively, for doubly- and singly-symmetrical sectioned members.

Thanks to the attention they deserve (or not deserve) but it took many decades for "Unsymmetrical Shapes" to be formally recognized. Advices on the treatment to the Unsymmetrical Shapes finally (not long ago) made its official debut into the modern Codes. More specifically as of this writing what as instructed in the modern editions of **AISC** could be summarized as follows:

(a) On flexure alone, *the Lateral Torsional Buckling stress per **Section F12.2** is to be determined by **analysis*** and

(b) On the combination of flexure and torsion, *the Buckling stress per **Section H3.3** is to be determined by **analysis***

The mandate sounded a bit fleeting for such a monstrous topic. But to clarify the obscurity (if any) on what does "**analysis**" really mean, one would need to turn to the **Code Commentary** portion on Section **F12** for further advice, but then could be overwhelmed by these statements that might vary in wording from different Code editions (unless phrased differently in different editions):

> "*The stress distribution and/or the elastic buckling stress must be determined from **principles of structural mechanics, textbooks or handbooks**, such as **SSRC** Guide, papers in journals, or finite element analyses. Alternatively, the designer can avoid the problem by selecting cross sections from among the many choices given in the previous sections of **Chapter F***"

Isn't that crispy clear or even more confusing after reciting the Commentary text? Or just as affirmed in the very beginning of this Article: "**There seemed more inquiries than resolutions on the issue**?"

Here are a few sample layman questions on unsymmetrical sections that could be equally puzzling to most rank and file Practitioners:

- For the subject of *stress distribution* and/or the *elastic buckling stress* as cited in the Commentary, how many different buckling modes should be identified or considered?

- Are stresses to be evaluated by the individual buckling mode or by combination with torsion? Does elastic buckling stress include lateral torsional buckling stress? …

- Does that insinuate the answer lies "somewhere" in the **principles of structural mechanics, textbooks and handbooks**? …

Understandably, most of these questions were framed up on random engineering intuitions, with a few appeared made up through quiet kind of silent technical protest. No matter how naive or how sophisticated one views in all regards, but in simplification, it might be advantageous for now to put a cap on all those unanswered or unanswerable questions, and concur with the **Commentary** advice dutifully. Other then that, there would not be better alternatives left but to just do it and see what it takes to determine the critical stresses by **analysis**.

In case of starting the **analysis** "from ground up" while in need of quick guidance on unsymmetrical sections' behalf, the first logical course of action before doing the analysis would be to command a "serious" literature search, followed by a "serious" review on the subject matter.

> It shouldn't take much effort to realize that there was indeed abundance of documented **R&D** results on buckling and torsion; but so unfortunately that "none" were very helpful for the design specifics on **CRG** with unsymmetrical sections. Under the circumstances one wonders:

- Does the **Commentary** sound like encouragement for more **R&D**?
- Does it imply a subtle form of in-my-face (or in-your-face) legal disclaimer or some sort?
- What are our options then?

Perhaps prior to fully digesting all the fine points/prints from various sources, we need to draw together our shared state of mind. *As of this writing,* the majority of Engineering Consumers who were in dire need of useful information seemed stranded with the dreadful feeling that there still exist some huge void/space between what were available from the updates in the current Codes/Design Guides and what were anticipated. Then as if not already apparent so far, but it is not that far off from the **non-R&D** realism for us to realize:

It appears the state of the art in **LTB** has not "officially" entered the realm of standardizing the "design" of open-sectioned members with unsymmetrical profile

Otherwise with due diligence by now for unsymmetrical sectioned structures, there should be serious development in the limit states of Lateral Torsional Buckling Stress "**as determined by analysis**" somewhere, somewhat and somehow by someone if not being kept in utter secrecy, or from credible source, has there been any?

As a result, one of the options left would be to perform some do-it-yourself kind of trial-and-error sessions continually, if not pioneering some forms of analysis in the lab-testing facilities or elaborating on pieces of (electronic) paper in that regard. But easier said if only we all have the proper (software) tools, testing (hardware) equipments, the technical know-how and the do-it-yourself-styled **R&D** urges, etc. and more notably if only the budget, billable or non-billable, would allow for such undertaking.

But predictably so, performing **R&D**-styled analysis at any level of sophistication in determining the critical **LTB** stress for a given **CRG** project of late involving unsymmetrical profiles would be a very tough-sell and a taxing luxury to most Engineering mainstreams. And even if the task is not technically controversial enough yet it would be a tougher-sell to the Facility Owners who are (not) willing to foot the phenomenal engineering bills because the task is too complex/cost-prohibitive for regular practice.

After all even with all good intentions, if there were (computerized) analyses done, it would still be difficult to justify for the cause that how any analytical model (or models) could be idealized with proper setups and obtaining satisfactory results that could closely emulate all or in parts of the realism, for instance:

- The actual **CRG** moving load application (not just the application of some concentrated loads at the mid-span of **I**-shaped beams)
- The realistic boundary conditions binding the support restraint objectives applicable to both flexural and torsion behaviors
- The true simultaneous **3-D** load application at offset distances from the **shear center** reflecting the actual operating conditions
- Handling the behavior of non-compact sections and justifying qualification of structure meeting fatigue design criteria, etc.

Yes one by one, execution through divide and conquer in that treating each task as if independent on its own worth might come to a piecemeal satisfactory ending. But for all that (and more) being combined onto a turn-key **CRG** application with unsymmetrical sections in general configuration, the "practical design" for which is neither an one-shot deal nor as simple as plugging numbers into a few formulas then calling it done but a very tedious trial and error progression.

Chances are that if ever failed in any one aspect (stress or deflection) on the first try, one would like to learn the reason why it failed − in meeting the stress/strength or serviceability criteria − before the geometric model could be improved on; and then even if the attempt succeeded, in the end one would also want to <u>learn</u> more such as which dimensions could be optimized for better performance, and so forth.

Being "practical" in any "practical design" session before drawing to a conclusion while going through with each of the various analytical "debugging" stages, Engineers should try finding answer to some simple but important questions, for example:

- How does anyone know from the output constituent which part of the longitudinal stress is due to flexure and which part of that stress is due to warping?
- How much is the lateral translation (displacement) at the railhead?

- What about fatigue shear stress reversal at certain bolt hole?

  With limited resources as in all modern (office) practices, searching for proper answer to these seemingly simple questions even for <u>symmetrical sectioned</u> members is never cheap on the labor side and/or never easy on the technical side. If finding it ever so difficult to provide straight answers to these simple questions for symmetrical-shaped **CRG's** sake then what could anyone expect from the analysis done for <u>unsymmetrical sections</u>?

As for any one-timed "analysis" being completed for a specific **CRG** assignment, if no one knows the (proper) answer to those important questions right off the bat or doesn't have credible means in optimizing or defending the analytical results, wouldn't that be a waste of that one-time effort (time and money) for not contributing to the growth in practical hands-on knowledge?

1.18 What Could Practitioners Do About Unsymmetrical Sections?

First of all, unsymmetrical sectioned girders are not just happened overnight and there were aplenty in many Mills since day one. Secondly to these structures, whatever information out there for many years such as (misunderstanding) their true behaviors or understanding the true cause of their in-service distress, etc. is still out there happening today or yesterday. Thirdly, much of the distress in the structures were evident (if not still hidden) from the observation per most recent field inspection on those troubled **CRGs**, and the majority of which were designed more than half a century ago.

But let us face it:

> These age-old technical bottlenecks, through slide rule, hand-held calculator and now into the computerized or internet-crazed era, would not go away and would still need to be broken through not only for benefit of all the future and existing facilities but also for the ongoing or future applications. Unless the Code Committees or Design Guide Providers act in time <u>to the point</u> otherwise, it will never help no matter what the existing (or the future) Code position is
>
> Again, for "practical" purposes, the Code Committee may have done the best it could in general, but only in accordance with recognized "engineering principles" derived through proven **R&D** efforts (then and now,) and therefore has declared that the Code is **for general information only**. What matters to the Structural Engineers "in practice" or in place like the court of engineering justice is asking seriously: Is that really just for **general information only**, and nothing else?

Let us face it, again:

> It doesn't matter whether if the design recommendation is <u>practical</u> or not, but the Code Committee although doesn't have to be binding to all comers, yet it may not have apprehended the fact that Structural Engineers of many fates in many camps had to deal with structures of all ages, shapes, sizes and conditions
>
> Some lucky sector has the luxury of designing "new" structures and of course can enjoy the "how to do it right" by the book or by the most up-to-dated Code intents. But meanwhile not so luckily that another sector has to struggle making do with all the "inherited" girder attributes and they're wondering "what's been done wrong long before their time" and "where is the missing book" that were meant for more than "**general information only**"

Then think about being stuck with upgrading the lifted capacity of an existing **CRG** of unsymmetrical sections and were told point blank by the Code **Section F12**:

> "*… Alternatively, the designer can <u>avoid</u> the problem by selecting cross sections from among the many choices given in the previous sections …*"

> How rationally could the dilemma if not be avoided but by doing so in the real world? Or does that sound more like “Sorry you **CRG** Engineers or Facility Owners try a new girder shape of symmetrical section; abandon the unsymmetrical sections or <u>you are on your own</u>?”

R&D can sometimes somehow somewhat promote or evade the urgency or priority on “when” to tackle the problem pending availability of funding or other technical breakthroughs. But time won’t let them the Mill Facility Owners in real life but had to go on with their refurbishing plans whether involving repairs and/or upgrades. After all, Engineers with no escape but were assigned to the unfortunate repair and/or upgrading sectors don’t have much choice unless they can <u>afford</u> in business sense to do it the same old off-the-mark way or abandon the project all together and not getting any reward or being paid, that is.

What long overdue to the **non-R&D** professionals who can’t wait any longer were some “<u>practical</u> interim alternative rationales” despite the nonexistence of authentic experimental or research data in backing up any of those alternative rationales if ever there were any.

Funny how not many dare to question some of those **old-timer** mentors, gurus, experts or even the “Design Guide Providers” on the specifics of unsymmetrical sections for afraid of being tagged as technical renegade, unconventional or plain disgruntling, etc.

But let us face it, once more:

> One of the major hung-ups, in the olden days and of late as well, could very well be rooted from misunderstanding in design of **CRG** against metal fatigue. A lot of those Old-timer Engineers, rampant in the Engineering society with so much influence for so long, often overpowered their younger peers authoritatively using outdated technical know-how pushing through their pseudo-expertise in **CRG** design. Facing these amateur-in-disguise that some of the **newcomers** felt like no real “books” to turn to even if they were so eager to learn solving problems the true problem-solving non-hocus-pocus way

By staying in a technical low profile and continuing practice by the Book, wrong Book or no Book and never defy the wrongs from rights or the rights from wrongs may be more wrong than plain wrong. And yet there should be no harm done if Engineers keep asking no-nonsense non-trivial questions and keep demanding and searching for reasonable answers then a rudimentary strategy, unofficial but practical for daily problem-solving purpose, might just unfold by itself until otherwise. **After all, the ball is in the air but it’s a long run for us to reach the home base and let us move the subject on further; this is only the temporary ending of the beginning**.