Open Sectioned Crane Runway Girders with Arbitrary Profile Geometry

Chapter 3 – Advanced Introduction (Part 2 of 2)

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3.1 Could Unsymmetrical-Sectioned CRG Experience Lateral Torsional Buckling?

While bringing up this somewhat puzzling question per se, it turns up another one:

> By the asking, does that (or not) make any **R&D** sense beyond our own common sense?

It'll be too soon for a straight answer without serious **R&D**-tailored contemplation. Then only on self-awareness purpose, a very subtle cross examination that may hit us not that hard but wondering, who really symbolizes **R&D**? Nevertheless, questing for a modest advice over such an exceptional issue on *unsymmetrical sectioned members' behalf* does seem fit as breaking through one of those technical bottlenecks mentioned at the beginning of **Chapter 2**, or is it not?

Anyhow, whether out of curiosity or pure interest, at some point or another, many Structural Engineering Practitioners would need an **R&D** signoff for it, just to be sure.

Knowing the fact that well-founded (or well-funded?) discussion on this matter was rare or rather vague for time being (as of this writing,) so let's see if there is any easy way out on our own to piece together the puzzle on *the design exclusively of unsymmetrical sectioned girders* by further posting the following;

- *How shall we deal with **Lateral Torsional Buckling (LTB)**?*
- *Do we really have to bother with **LTB** issue(s) at all?*

Or so for a much wider coverage, those two question(s) might be consolidated into something like this:

> *Should the occurrence of **LTB** be fully dependent on or selectively be independent from the profile geometry at all?*
>
> Perhaps it can go either way, but here's the main reason why bringing up the subject of interest:
>
> As explained in many textbooks as we'd learned, **LTB** obviously pertains to symmetrical sectioned (mostly **I-shaped**) members, but the practical notion on **LTB**'s happening (or not happening) to **non I-shaped unsymmetrical sectioned members** seemed somewhat tentative or otherwise hesitant if not consistently missing

Beyond **I-shaped** members on Practical Structural Engineering Rationales, the need of clarification and advice on **LTB** were very specific: As we normally think, it should apply to all sizes and configurations of **Crane Runway Girders** (**CRG**); or does it?

At this point even though there isn't much inspiration on pressing a sophisticated engineering logic (as we would start forming the not-so-sophisticated judgement a few pages down) to the issue, but the absence of such an interesting (important or not) subject has to be cracked open somehow through some means – without reinventing the wheels but drawing on reliable resources already existed out there − *since there* *weren't "Official **R&D**-blessed words" on this subject in the short run so what is about to transpire next should be assimilated with a grain of salt*.

3.2 Revisiting Principal Axes

First thing first, *let **Y-load** be pointing along the gravitational axis* as always. It is important to point out the **Y**-axis as we speak of is the one that matches the loading axis whence the orientation it points to might not match the innate principal **Y'**-axis for unsymmetrical section.

To simplify the loading scenario and treatment on matter of concern, one could put aside the effect due to longitudinal force by letting **Z**-load ≈ 0 for now thus leading to the most basic **XY** loading condition; further on if also letting lateral thrust **X**-load ≈ 0 then the **CRG** is mainly influenced by (1) vertical **Y**-load, along with which (2) a companion effect from $\mathbf{P_y}$-$\mathbf{\delta_x}$ torsion − **Mz** (or **Mt**) − due to *misaligned **Y-load** eccentricity* $\mathbf{e_x}$ with respect to not the girder web but to the **Shear Center** (**SC**).

> By completing the free-body diagram based on *global* static equilibrium, a *local* cross section can be shown to undergo:
>
> - Flexural bending moment $\mathbf{M_x}$ – bent about user-defined centridal **X**-axis (perpendicular to **Y**)
> - Flexural transverse (**Y**) shear
> - Pure torsion $\mathbf{M_{z0}}$
> - Warping torsional moment – bi-moment – $\mathbf{M_{zw}}$

Prior to venturing much deeper into the issue of **LTB's** *happening-or-not-happening* to Unsymmetrical-Sectioned Members, it should make things easier later on to first refresh the (See **Chapter 1**) characteristic of **CRG Reference Coordinate System vs the Principal Axes System**:

> Very likely in most practice sessions from not knowing the whereabouts of the **true Elastic Centroid** (**EC**) at the beginning, the usual setup of conventional **X/Y/Z** axes for *unsymmetrical sections* by Users (us the Engineers) would have located the origin of "reference system" somewhat *randomly* with orientation of axes be more *consciously* to match:
>
> - Either the orientation of one of the major component(s) of **CRG** section
> - And/or the dominant universal global load senses directing at the crane railhead

By such *hit-or-miss* setting of the system origin, a situation is to be expected as follows:

> For systems not explicitly catered to *doubly symmetrical-sectioned members*, the **chosen X/Y axes** for *unsymmetrical section profiles* would stand a *very slim chance* to match up with the orientation of **true elastic principal (X', Y') axes** in the first place, let alone overlapping in one-to-one correspondence − unless all the so correctly **chosen** axes-vectors in conjunction with the pinpoint nodal coordinates of the **true Elastic Centroid** were:
>
> - Pre-confirmed with one another and
> - Prearranged or rearranged purposely based on proper calculation performed ahead of time

Consider a dire interest in the numerical treatment to various constituent elements that made up the "geometric property domain" of **unsymmetrical section** in general:

> Prior to starting serious *derivation, formulation or calculation* of specific property further than *cross sectional area*, it is important to explore every object element with **3D** perspective thus to help recognizing that the **vector orientations** *drawn* for the *chosen* **X/Y/Z** and that for *true* principal **X'/Y'/Z'** are most likely independent to one another
>
> > A notion by sharply defined opinions:

> ***X/Y/Z*** *is akin to a users' scheme arbitrarily* ***chosen*** *based on intuitive convenience while* ***X'/Y'/Z'*** *is mathematically confirmed and defined, which is* ***distinctive and unique*** *in all geometric section shapes be that symmetrical or unsymmetrical*

The clarification as expressed might inspire a practical (but not mandatory) repositioning of our mindset, which might not seem much weighty in applications elsewhere, but in **CRG's** importance, it should fit in appropriately as soon as we advance into an inherent design condition − of no escape – there unmistakably lies an *unsymmetrical sectioned members' deep-seated challenge coming from flexure and torsion both the* ***same time*** *and at* ***all time***

There are good reasons to pay closer attention to the much *generalized non-agreement* of axes *drawn* between **X/Y/Z** and **X**'/**Y**'/**Z**' especially with aims of:

- Better handling or understanding of the (probable) *variations* in profile geometry connecting (*gross*) pre-buckled section and (*effective*) post-buckled section

- Benefiting from much simpler numerical workout as to computing conventional structural responses to loads and/or internal stresses with less hassle through using fewer parametric terms mandated in most algebraic (equations) expressions, etc.

So every now and then, we need to let go of and free ourselves from total dependency on situation as if being stuck exclusively with **X/Y/Z** convention − in other words, maintaining an **X**'/**Y**'/**Z**'-oriented mentality for as much or as needed as we could − now becomes more imperative

Such a rebooted **X'/Y'/Z'-oriented** mindset does make better sense in case some of us not yet accustomed to the fact that it is more clear-cut and more practical to deal with principal axes **X'**, **Y'** and **Z'** − yet when in need, which can always be converted back and forth with **X/Y/Z** system through proper coordinate transformation (implicating translation and rotation)

However, the engineering principle based in execution of many *must-do normal routines* is **practically similar if not all same** *to all outward appearances* whether opting for **X/Y/Z** or **X'/Y'/Z'** only when (1) studying elementary structural behaviors 101 by the Books or (2) handling basic structural response to basic loads by Design Specifications, etc., **except** *when dealing with tensile stress fluctuation and shear stress reversal or strength provisions keen on qualification of unsymmetrical sectioned* ***CRGs in general***

Crane Runway Girder takes on global load at the rail top as a resultant − duly applied at interface where crane wheel meets crane rail – or that being resolved into vector components pointing into independent senses either along **X/Y/Z** or **X'/Y'/Z'**. When judging their influences to **CRG** from among all load constituents and orientations; we understood:

> By numerical measures in normal practice, the impact to design outcome – in term of rendered cross section configuration – attributed to the sheer magnitude of vertical **Y- (or Y'-)** load is much more dominant than that out of **X/X'**- and **Z/Z'**- loads (regardless if loads were passing through section's **Shear Center** or not)

Y-load deserves much higher-priority attention for obvious reasons:

> All so not only for its dominance by way of definitive magnitude imparted but also for being the most natural entity among all load sources – for it (1) is largely well-defined and (2) always fixes earnestly along or next to the referential gravitational axis by nature. Then when time comes to converting flexural response due to **Y**-load from **X/Y/Z**-based convention into **X'/Y'/Z'**-based convention, one needs to perform simple transformation such that:

- The internal moment **Mx** becomes two component projections **M’x** and **M’y**, likewise
- The internal **Y**-shear turns into projections into principal **X’**-shear and **Y’**-shear

Transformation as simple as it may seem but in the process, one needs to (1) always **watch out for proper sign conventions** and (2) prepare for effects with respect to other 5 degrees of freedom

*What if one chooses to do without **X’** and **Y’** and stick with **X** and **Y** then what could happen?*

Per basic Engineering Mechanics on calculating Flexural Bending Stresses for instance, one has to cope with using convoluted formulas involving non-principal-axis-based (1) moments of inertia **Ix**, **Iy** along with (2) product moment of inertia **Ixy** in order to arrive at the correct result agreeing with “proper” flexural response to loads

Nevertheless, principal axes exist as “natural” and as “practical” as can be that shouldn’t be stowed away, be forgotten about, ignored or misunderstood or be mixed up with other definitions and purposes.

One of the most common mistakes:

Applying geometric properties without distinguishing the difference between user-chosen **X/Y/Z** system and the principal **X’/Y’/Z’** system especially not paying attention when:

- *Calculating **MC/I** type flexural bending stress or **VQ/It** type of flexural shear stress* or
- *Locating **shear center’s** whereabouts and/or calculating **warping related properties***, etc.

Just picture the most familiar setup of **X/Y/Z** for doubly symmetrical sections the usual way, of which very seldom or never was there a need of calling out or referring to principal axes by name for obvious reason:

For doubly symmetrical sections, the customary **X/Y/Z** system per intuitive convenience would have been chosen as identical to the innate **X’/Y’/Z’** unless deliberately made them at skew or offset with one another for some oddball reasons − yes, odd if someone actually does that

Therefore ordinarily for doubly symmetrical sections’ sake, one would take the no-brainer advantage that simplifies the geometric reference with respect to only the **elastic centroid** (not just by luck but mathematically which is also the **plastic centroid** and **shear center**)

Judging from the level of difficulty in handling engineering mechanics matters, obviously: The “referential amenity” offered by doubly symmetrical sections is at one extreme while the section geometric “referential inconvenience” for unsymmetrical sections is at the other extreme

*Thereby in normal practices no matter how inconvenient that is and whether the structural member was loaded with torsion or without torsion, all section properties should be clearly identified and calculated in regards to both **X/Y/Z** and **X’/Y’/Z’***

One of the serious pitfalls to avoid while dealing with unsymmetrical sections:

It would only be worse (meaning rarely or never be better) whenever the “insensitive” ones among us did “forget” to resolve the load influence from **X/Y/Z** into **X’/Y’/Z’** or exclusively made use of geometric properties based on **X/Y/Z** system for all calculations; what could happen then?

Sounds fictional but factual:

*Just be wary in case(s) if carelessly from mixing up **X/Y/Z** with **X’/Y’/Z’** , the true value of certain critical stress could be ten, twenty times or more in error on the unsafe side depending on how “unsymmetrical” (or how irregular) the cross section geometry might have dictated*. ***Yes***, again, *just be wary*

3.3 Get Going by Engineering Judgement?

Authenticating whether the ever-bewildering event of **Lateral Torsional Buckling** (**LTB**) could (or could not) happen to *unsymmetrical sectioned* ***CRG*** might need a whole lot of *intellectual statistics* as backup.

Provided it is true that:

- **LTB** were proven to be factual (in theory) and could actually take place in real time, and
- It is of **R&D**-promising as to describing such puzzling event mathematically

If these provisos were valid then the technical undertaking would likely to engross a multitude of variables and parametric clusters no less than the mixture implicating *bi-axial bending,* ***St-Venant*** *torsion, warping torsion, lateral restraint spacing,* ***Elastic Centroid****,* ***Shear Center*** *and so forth*

Imaging in *structural-mechanical* sense:

The complexity as previously revealed in the notorious *equation or formulation* for *symmetrical-sectioned* **LTB** (already in the Books) offered ample hints regarding the level of sophistication required to delve into *unsymmetrical*-sectioned **LTB** (if exists in theory) − although herein the situation remains unsettling; yet it would be way beyond reasonable ingenuity on what the *simultaneous* ***FTB*** *differential equation series* may look like, let alone coming up with any viable solution for which – now the question for practical engineering purpose: *Is it really worth it?*

(Trivial) **LTB** opinion on unsymmetrical section:

It supposes that the terminology for this distinctive buckling mode, if proven existed for unsymmetrical sections then it should be much more descriptive than the simple phrase **Lateral Torsional Buckling** − could be identified as "Hit-and-miss Lateral Torsional Buckling," "Unsymmetrical Lateral-Torsional Buckling," "Unsymmetrical Torsional-Lateral Buckling" or a term unique to tell apart from symmetrical sections' **LTB**. Anyhow it's better leaving the "naming right" for mainstream **R&D** to settle. But to envision what may transpire to **CRG**, it appeared that the Design Practitioners could only resort to their own **Engineering Judgments** for time being.

3.4 The Die Hard Shear Center

The predicament from all indications was loud and clear that unsymmetrical sectioned **CRG** is always under torsion − even under its own dead weight. **Presumably** then, no matter what sources the torsion were originated from (whether **X**-load, **Y**-load or **P-delta's** or their combinations) and regardless to the magnitude of induced torsion, by which one might make a reasonable deduction as follows:

Once an ***infinitesimal*** *rotation is made active from* ***Dead Load*** *and the minute as the cross section is under mechanically induced live loads' stimulus from the rail top, it tends to (1) either furthering (2) or counteracting the rotational movement about the principal* ***Z****-axis − objectively through the* ***Elastic Center*** *− prior to settling into a final yet "unknown/not-yet-established"* ***goal orientation;*** *and it doesn't matter whether the pivoting is initiated clockwise or counterclockwise but it should depend on:*

- *The orientation of the initial live load resultant*
- *The torque directive with respect to initial reference –* ***Elastic Principal Axes*** *and*
- *The relative offset between* ***Elastic Centroid (EC)*** *and* ***Shear Center (SC)***

All that making any sense or not remains to be seen but it should

Some may still be wondering why bother with **Elastic Principal Axes** in interest of **LTB**:

Imperative as it is from being reminded to hang on dearly with **Elastic Principal Axes** because mathematically and natively these axes are stuck permanently with all cross sections. Keep in mind that *the most secured or the most stable axis of bending of any cross sectional geometry against **LTB** is the pivot about principal minor axis **Y'*** (or whichever orientation about that the principal moment of inertia has the minimum value among all domain-wise numerical values)

In other words while considering only flexural influence, if the applied resultant flexure bending moment happens to be bending about the principal minor **Y'** axis then we would have no worry about occurrence of **LTB**

*Wouldn't it make perfect sense to differentiate that **(Y')** principal weak axis is indeed our **goal orientation** to settle in just as pointed out several paragraphs back?*

In an idealistic and realistic world of unsymmetrical sectioned **CRG**, if concentrating on the effect due to only the most dominant load, **Y**-load to make a point:

Again ideally, in order to accomplish the most optimum and the most natural settlement into the not yet defined ***goal orientation*** − through transitional stages − it would wind up with a situation as if the cross section is coercing the principal minor **Y'**-axis into overlapping with:

- Either the global **X'**-axis only by chance, so long as for which the principal moment of inertia **I'x** is of the absolute minimum amount – **but**, this is *realistic* only for *circular shapes,* which as understood is immune from **LTB**

- Or – equally *unrealistic* − the orientation perpendicular to gravity axis (which is the *orientation of the dominance of **Y**-load*)

Obviously neither condition is viable in normal practice; now comes the realism: As expected for any *unsymmetrical-sectioned* **CRG**, the global **X**-axis chosen (by user convenience or not) for whatever causes, which is not one of the principal axes and it might never be that lucky to hit it right on

Should all as-said is not convincing enough then take a closer look and see what as-staged as follows if it makes better sense:

*It all begins with the structural member being restrained at both ends against **Y/Z**-translational movement yet see what happens if letting the ends be:*

- *Free to slide* laterally *along **X**, and*
- *Free to* rotate *about the **Z**-axis*

*For both ends are free to move along **X** and rotate about **Z**, this would lead the way to sliding and rolling over − not quite an unstable situation yet but could be worse – thus the ends must be properly restrained against all X/Y/Z movements*

*In this case in order to prevent **LTB's** happening, the ends must be artificially anchored in such a way that would finesse the load resultant (1) to pass through the **shear center** and (2) into bending about the as-said one-and-only Y'-axis or **goal orientation**; in other words, as soon as girder ends were held firmly against twisting, the **shear center (SC)** of the cross sections elsewhere (other than the support nodes) would immediately **seize control** to become the "**center of twist**" albeit the rotation is initiated about the **elastic centroid***

Consider an interior **X/Y** node at some **Z** distance away from the ends and provided the member, under torsion, was furnished with properly detailed/fabricated boundary connections given ample rigidity to fulfil the supporting functions, from which theoretically there could be two **extreme outcomes**:

(a) In due course, the **Z**-rotation about the **SC** would reach its upper bound and then come to a stop at global equilibrium. But such equilibrium condition could realize only if the torsional stiffness is (substantially) rigid enough to curtail any further rotation from that point on

(b) Or else, the (angle of) **Z**-rotation would continue to grow; as it increased beyond what the elastic limit of the *local cross section(s)* and/or the *global supporting system* can tolerate, it eventually led to an inelastic or unstable state, or a total collapse

Up to this point short of a total collapse, all sounded reasonable in theory (or made believe by pure imagination) but the genuine concern actually lies somewhere in between these two extreme outcomes just as pointed out. Then one can raise a question:

> *Would the **CRG** fail prematurely somehow due to local yielding and/or local buckling prior to reaching a torsional equilibrium, with or without **LTB**?*

It probably would; but, one needs to clear up the thread of thought and go back to where we were from a while back as picked up in **Chapter one** on symmetrical sections' behavior:

> Recalling in which, the torsions of interest were not inherently inborn but resulted from one of the two global events; (1) **Lateral Torsional Buckling** or (2) **Flexural Torsional Buckling** (**FTB**)
>
> That is to say: The torsions were induced via external means attributed indirectly from detached sources: (1) either from the strong axis bending **Mx** owing to **Y**-load passing through the web centerline or **shear center** for that matter, (2) or due to an axial **Z**-load regardless to whether it was applied off-centered with respect to the **elastic centroid** or not. Remember warping?

For unsymmetrical sections, let us bring up some noteworthy circumstances:

> In addition to taking stimulus from flexure − the uniaxial and/or bi-axial bending − torsions were already a part of the load originators *unless* the resultant from all loads (including dead weight) were applied (or resolved) purposely through **shear center**; notice the crafty use of word *unless* is not very friendly as far as unsymmetrical sectioned members are concerned
>
> Therefore in this instance, torsions came **actively** through "external static force equilibrium" due to **P-delta** and were not being **passively** put on. It is different from the case as for symmetrical sections that involve "compatibility in deformation" owing to post-buckling (or at buckling whether triggered by **LTB** or by **FTB**)

Here is another "challenging question" that may be beyond pure interest of many Practitioners' if not already part of our immediate concern:

> *Could passive torsion be brought on by **LTB** to unsymmetrical sections just like it could do to symmetrical sections?*

The question is still out for an Official Response. It remains to be seen (heard) wherever there is credible settling of a "mainstream **R&D** answer" on behalf of all Structural Engineers; but at this juncture we ought to start and/or end somewhere without the blessing.

3.5 Interaction among Longitudinal Stresses

Making it simpler hereinafter by setting the shear stress aside while digging much deeper into the longitudinal stress domain then "see" what's in it altogether:

> First thing first, same as those well-understood flexural bending stresses designated $\pm\mathbf{f_{bx}}$ and $\pm\mathbf{f_{by}}$, warping normal stress $\pm\boldsymbol{\sigma}_{\mathbf{n}}$ also comes in two flavors:

Either tensile or compressive, all were pointing along the local **z**-axis as well

Visualizing within the confine of a specific **Z**-cross sectional slice subject to bi-moment $\mathbf{M_{zw}}$, for which some might wonder what could be sketched out of such "out of the ordinary" effect in graphical sense:

Just take a look at the mapping, plotting or the charting of warping normal stress − qualitatively − if $\pm\boldsymbol{\sigma_n}$ of every nodal point speckled all over the **XY** section profile were *enveloped* all at once then, what stood out the most from the topography depicting the stress dispersion are two pairs of *Peaks and valleys* – yes, two pairs

The measure of each and every element subsists in the $\boldsymbol{\sigma_n}$ domain is hinged on (1) unique **X/Y** coordinate and (2) correspondingly with which the nodal unit warping, $\boldsymbol{\omega_n}$. By calculation, $\boldsymbol{\sigma_n}$ would have to follow the native sign convention and the numerical value came out of the formula $\mathbf{M_{zw}} * \boldsymbol{\omega_n} / \mathbf{C_w}$

The fact drawn out of that formula – or that mathematical expression so to speak – is, the quantity of $\mathbf{M_{zw}} / \mathbf{C_w}$ could carry either a positive or negative sign, which is inherent from $\mathbf{M_{zw}}$ in turn being passed on by global load analysis

At any rate, the value of $\mathbf{M_{zw}} / \mathbf{C_w}$ bearing a sign remains unchanged across a specific profile is a constant not just for one single node but throughout every dot scattered within the **XY** profile slice's confine as well

Clearly then, the final nodal unit warping $\boldsymbol{\omega_n}$ "value" and the "sign" it carries along with would be wholly responsible for the ultimate distribution of $\boldsymbol{\sigma_n}$ – that goes to show the importance of maintaining proper sign convention under control *in every move*

With $\mathbf{C_w}$ (1) being *positive definite* at all time and (2) by funneling the warping stress formula into two *sign-controlling* entities − $\mathbf{M_{zw}}$ and $\boldsymbol{\omega_n}$ − as we just pointed out is not by any means very high-tech in concept at all; but what being identified is quite important for better qualifications of these facts:

- $\mathbf{M_{zw}}$ is exposed in the global front *actively* at specific **Z**-coordinate

- $\boldsymbol{\omega_n}$ is hidden *passively* as local geometric property from within the spread of **XY** plane corresponds to that **Z**-coordinate

To some Readers, all that being called attention to seemed utterly redundant and might not worth spending time with; but then unless we make sure everything and all things were done correctly and properly or else (1) during a design "debugging" session suspecting that something might have gone wild in middle of certain numerical step, or (2) from not knowing any other better way out of the **numerical trap** in that, one could easily be led off course from a four-way mutating between $\pm\mathbf{M_{zw}}$ and $\pm\boldsymbol{\omega_n}$ in, which is critical during evaluation of *fiber stress reversal*, etc.

Lesson to be learned:

To avoid **numerical glitches** when evaluating **CRG's** adequacy node by node per fatigue assessment mandate, $\boldsymbol{\sigma_n}$ owing to each **P-delta** source should always be registered with *proper sign(s)* prior to combining with the coexisting longitudinal stress due to other load sources (flexural effect triggered from pertinent **X/Y/Z** loads, that is)

The point:

Whether or not in agreement with what affirmed in the last few subsections, the combined states of longitudinal stress in **CRG** had been proven a rather "complex mess" (by the same token a similar mess also construes to the shear stresses also) as we have seen or soon shall see

So either playing by the rules shrewdly or not, Readers in certain easy-going faction might envisage taking haphazard shortcut (but don't do it) or letting the fiber stress/load combination process go off somewhat too easygoing, by all that only so fortunate if the **CRG** does not suffer from metal fatigue (who knows when it will?) in a long run, nevertheless taking it too easy is not a good start/end as to meeting quality assurance intent

The naysayers may all be disagreeably stubborn at this point yet not until they actually saw in real life a much worse case − catastrophic-like incidence − such as base metal cracks formed across the entire length of the flange top and/or the cracks extended down the girder web or for the worst as the entire overhead crane being driven into the ground, then what?

Further understanding of the "complex mess":

Pick any isolated global **X/Y/Z** locale, for which the instantaneous state of **longitudinal** stress along **local-z** is always a mystery at early stage of engineering process, i.e. the "numerical ambiguity" will linger on until the *aggregate of each participating stress variety* was consolidated (or enveloped) from all pertinent effects wrapped up from **all moving load cases**

The disparity in the state of stress from one **X/Y** node to another **X/Y** node − even not that far apart but on the same **XY** plane – could remain the same or else be branded as either qualitatively null or in contrast unconditionally enormous. Thus in general sense and in essence the stress resultant would fluctuate closely with the "**numerical chemistry**" as hidden or as exposed in the state of each and every aggregate variety within the longitudinal stress domain {$\mathbf{f_a}$, $\mathbf{f_{bx}}$, $\mathbf{f_{by}}$ and $\boldsymbol{\sigma_n}$} – i.e. consolidated from both flexure and torsion

But what does it mean by **numerical chemistry**?

First of all, **numerical chemistry** has nothing to do with the Pure Science of Chemistry but rather a convenient phrase happened only in this Article as to expressing the unpredictable and dynamic nature of situations we were dealt with

Secondly, at any **X/Y/Z** node, consider for which (1) during any calculation step/session and (2) for a specific brand of nodal stress of interest, we could and should anticipate two categories of **numerical chemistry** working in association (pair) to churn things up microscopically in some way − one comes from local section geometry (**I**, **c**, **Q**, **t**, $\boldsymbol{\omega_n}$, etc.) and the other from *enveloped* global load response (**M**, **V**, $\mathbf{M_{zw}}$, etc.)

Although the partaking parameters may be strategically *mixed, grouped or matched up* as "customarily" called for in various formulas for purpose of computing specific variety of stress, but each unique "class of stress" would always be engaging in a unique combination of **numerical chemistry** (dynamics) association of each own

> For instance, as in computing flexural bending stress as obvious as applying **M c / I**, other than the important role preempted by the local **X/Y** node-specific geometric properties, i.e. **c / I**, the more violent **numerical dynamics** at any specific **Z**-coordinate in focus actually subsists in the governing global **M** value(s) as far as **M c / I** is concerned; soon we should see why

At any given **X/Y/Z**-coordinate, prior to figuring out the final "extreme" (positive/peak and negative/valley) magnitudes of **M c / I**, the governing (range) value of **M** at that **z**-station must be determined first. ±**M**, at either numerical extreme, is nothing but a solitary numeral (usually from the **<u>enveloped</u>** moment diagram.) Accordingly on principle, what it takes to arrive at the critical bounding value(s) of **M** for design consumption should be fairly easy; isn't it?

Wrong, it is not that easy; –

> In fact the "extreme" (positive and negative) magnitudes had to be the collective effects enwrapped from all participating wheels/loadings moving *backward and forward, here and there and on and off* the girder, and by all means that's the way how load instances were identified – that is how **<u>enveloped</u>** moment diagram came into the picture
>
> And then if tracing all the way back to the root of each given load case and its unique load application nature such as *wavering in load magnitude, changing in **Z**-location and load sense, presence of impact effect and the frequency or cycle count of loading-unloading instances*, etc., regardless if some or all of those loads/effects had already joined hand into action physically or not but logically, all probable scenarios should be played out and taken in in the calculation sessions; that is to say whether the load is from <u>active</u> source or passively <u>passed on</u> from/into a secondary effect, all that could have been in a state of intensifying, receding, reversing, diminishing, disappearing, reappearing or repeating, on and on, etc.

See the looming data handling issue hidden behind? Proven already – what a big mess to emerge from for such small task!

3.6 The Inseparable Links between Shear Center and Longitudinal Stresses

Despite the big "mess" from sorting out the numerically induced ripples through sign permutation feats while summing up the nodal aggregate $\pm\mathbf{f_a}$, $\pm\mathbf{f_{bx}}$, $\pm\mathbf{f_{by}}$ with $\pm\boldsymbol{\sigma_n}$, one could take a diverse notion on how does *warping event* interact with *flexural bending* as follows:

> First, consider any **x/y**-cross sectional slice under uniaxial flexure bending, by which the longitudinal **z**-fibers were either stretched or contracted by tension or compression, respectively. Under strain equilibrium, the resulting *gross net axial strain* <u>integrated</u> across the **xy**-plane can be demonstrated that positive sum and negative sum would cancel each other out becoming net zero
>
> Next, add a global axial **Z**-load to the member – provided that does not instigate material yielding or stability failure – depending on which direction it points to, all local fibers would be stretched further or be shortened accordingly in unison, but still, the gross net axial strain across the **x/y**-plane should remain uniform or leveled − again by equilibrium – in a way as if **Z**-load merely contrived a catalyst effect to the existing uniaxial (or bi-axial) flexure bending
>
> And then, warping should come to equilibrium in its own unique adaptation independently only a bit more convoluted:
>
> > Warping normal stress $\boldsymbol{\sigma_n}$ across the **x/y**-plane could be (1) *dispersed* as tension or compression and (2) *in magnitudes* seesawing <u>unevenly</u> from a pair of maximum <u>peaks</u> to a pair of minimum <u>valleys</u>
> >
> > Due to <u>uneven</u> distribution of $\pm\boldsymbol{\sigma_n} = \mathbf{M_{zw}} * \boldsymbol{\omega_n} / \mathbf{C_w}$ across the local **x/y** plane, it renders a swollen/sunken kind of "warp-like" look as clusters of **z**-strain of identical sign were "congregating" by themselves within one of the distinctive *local profile divisions or unequally divided zones*, the featured node that stands out where $\boldsymbol{\sigma_n} = 0$ in each zone would share the same focal point at **shear center**
>
> **Equilibrium** across the section profile:

As $\pm\boldsymbol{\sigma_n}$ varies in magnitude from node to node with sign alternating from zone to zone in their own unique pattern; so is flexure bending stress as well having its own demarcation divide, only then the difference is in the "planeness" of deformed sectional profile; which remains *plane* under flexural bending but comes to be *distorted* under warping

Once warping strain was blended in with flexural counterpart, all highs and lows would consolidate into net zero warping strain as summation across the section considering the "gross" effect <u>integrated</u> (numerically) under **elastic equilibrium** condition, or else there won't be equilibrium

One may *look on the seesawing between "minus" sign and "plus" sign carried by warping normal stress* $\pm\boldsymbol{\sigma_n}$ *as "**catalyst**" that could somehow either <u>enhance</u> or <u>mitigate</u> the probability for the cross section to ultimately succumb into a (full-blown?) buckling failure*

However, depending on the "quality and quantity" of "catalyst effect" realized locally within each zone, where warping may or may not act as an effectual "catalyst agent" as perceived in some cases, i.e. $\boldsymbol{\sigma_n}$ may work out in moderating the **compressive strain** in certain critical nodes, quadrants, pockets or local zones, etc. or else trigger local yielding where the allowable strain cumulated at certain *extremities* had already been expended up into a state too close to the threshold by flexure alone − with very little or not much margin left

When there is not much or nothing left to share, the cross section has to do a global side-stepping into the direction associated with weakest resistance; *but, the caveat is once it moved side way or any ways to start,* ***shear center*** *would immediately come to regulate the pivoting motion*, in such way isn't this an unsymmetrical section styled **LTB** in display?

Under the worst scenario, as follow is only a wild speculation at this point:

Should anything "really bad" emerge during all these *enhancing* and *mitigating* activities accrued between flexure and warping, the compressive fiber stress/strain in certain elemental strands could already be way too excessive in the first place, let alone allowing an add-on at these strands to take further penalizing from repetitive fluctuation beyond the *threshold fatigue stress range*. Therefore a **local buckling** could be brought about prior to a full-blown **LTB-like event** (only if we prefer calling that **LTB**) whether if such phenomenon technically exist or not for unsymmetrical sections

Now deliberate the question:

*Would local buckling take place (much) sooner than **LTB**?*

Very likely the answer is "Yes" unless we made a point to prevent that from happening in the first place. For that we might accomplish through a few defensive moves against local buckling as follows:

(a) By constructing a "compacted" cross section profile complete with *fully compact* or *partially compact/non-slender elements*. In other words, by using relatively stocky **CRG** components to minimize the potential weakening in cross section's compressive strength against **local buckling**, or

(b) If already given a cross section with relatively thin slender elements prone to local buckling then we should "logically" discard the portion that is presumably buckled away under compression

Practicing with asserted defensive repositioning "by theory by the Book" by as-said should have the local buckling issue squared away (or precluded we should say.) But it's not 100% fixed yet; only then "in practice" there could be some other issues needed attention:

As external load increases, so would internal stresses; before long the resulting **fiber stresses** at the <u>extremities</u> may give in to local yielding, if not locally buckled yet

At one moment or another to a certain **X/Y/Z**, anything could happen:

> It doesn't matter if local yielding event takes place at a moment sooner or later than local buckling event or none of that at all, but at the least we need to do our best to prevent yielding and local buckling from happening given that **LTB** may not be coming at us in the rearview mirror yet

Here, a notion on **unsymmetrical sections** as some Engineers may view or review it from a different perspective, which is just as remarkable as we come to thinking it through:

> Again, take any **XY** slice along the girder length, once the **CRG** profile starts rotating about the **EC** (under any **X**, **Y** or **Z** load influence) the **SC** immediately interferes and takes over as the "*center of twist*" as understood. It then forces the profile section to undergo bending, shearing, localized stretching/contracting, twisting and warping at the same time and at all times
>
> What might follow then?
>
> A **full-blown torsional blitz** is instigated − with warping − perhaps not fully materialize for every application but at least it does in the world of **CRG**. In a way that warping would (1) act as an accomplice to further punish (distort) the cross sectional plane more than what has already been punished by flexure and would (2) stick along to interface **seamlessly** with the flexural responses

Notice that a very subtle "before-and-after" situation should become evident whether prior to or right after the **full-blown torsion** activated confrontation:

> It doesn't matter whether the "structural deformation" has potential to induce buckling or not but so long as the **CRG** is under load, in respond to which, any rotational movement departing from the "norm" is always resolvable via simple trigonometry into two vectors orthogonal to each other, or purposefully being resolved into one of the tangential displacements that is perpendicular to a certain polar radius as soon as the out-of-plane deformation/movement was captured – not by the **Elastic Centroid** but – by the **Shear Center**

And so as proven the local cross sectional plane is always under the spell of torsion or being rotated about the **Shear Center** except at the support points provided those were firmly anchored against rotation about the **z**-axis. All that then becomes a head start into the next setting:

*The rotational instinct of unsymmetrical section is so strong and so dictated by the **Shear Center** and it seems that lateral torsional buckling (**LTB**) doesn't even stand a chance to show up or sustain (practically lasted long enough) on our radar screen, makes sense?*

It should be fair to rationalize for *unsymmetrical sections*:

The seamless interfaces that exist between flexure and torsion are all natural regardless to:

(a) The load orientation and load magnitude
(b) The loading **Z**-locations, and more importantly
(c) Whether if lateral buckling (or any other mode of buckling) is **imminent or not**

Knowing that as load continues to increase, (1) the axial strain would build up further through flexural responses and accordingly (2) so would warping strain from the coexisting bi-moment, and then:

- **To a point theoretically** and ultimately, it might lead to the onset of yielding and/or (**lateral** or **local**) buckling from upsurge of local axial strain, which might not be very extensive but imminent at certain unsupported element/components' **X/Y** extremity node(s)

- At the **some (split) instant**, the instantaneous change in the inborn "coupling" of warping normal strain would act as if to "arrest" the member from further sidestepping and "coerce" it into conformance to whatever the state of "final" flexural-torsional deformations attained at **equilibrium**, so long as the member, free from local buckling, is:

    - Properly supported (anchored) along **X/Y/Z** at both (support) ends and
    - Stiff enough torsionally to regulate such conformance

- Or, the node/pocket/zone/section would fail by yielding long before anything else

<u>It comes down to the asking:</u>

Would unsymmetrical sectioned **CRG** experience **LTB** at all? If it does and only if it wins over torsion then in theory (what theory?) the answer is "Yes" except that no one knows for sure yet but Practitioners need to know:

(a) From **R&D**, whether it is a very short-term event or else strictly a linear elastic problem, non-linear problem or mixed, and

(b) For practical design check, how to derive a conservative value of **Fcr** for plugging into the **AISC Eq. F12-3** (as of this writing) or being conservative or not, the design qualification has nothing to do with **Fcr** is load resultant does not pass through **Shear Center**, does it?

For being short of clear-cut advice from official source for time being, we are on our own to carefully *verify, quantify, qualify and justify* all that we do − right or wrong − with backup calculation.

3.7 What Does LTB of Unsymmetrical Sections Look Like?

If **LTB** ever takes place so pertinent to unsymmetrical sectioned girders then one must ask:

> Could anyone authenticate a claim as to witnessing an actual **LTB** in the <u>dynamic act</u> and recording the moment it did and also how long it had lasted or what does the aftermath look like, or they think they did?

No matter what answers were given at this point, chances are it never did; so on conditions it did happen then, where is the backup number and of what measure? Yet only so proven if (yes, only if) putting faith in the enumerated information per documented findings collected through (detailed) structural inspections, observations, videoed sightings and survey results or scanned data, etc. some of which in steady state appeared to suggest evidences in form of permanent deformation thus with no official endorsement, in this regard as if **LTB** had actually materialized, Really?

Let's say it happened; on the aftermath of **LTB** or the phenomenon of post-buckling/buckled, if a typical finding of that were trusted as closest to being a genuine **LTB** then naturally such sightings should be more pronounced in the longer-spanned girders, especially those having relatively **weaker** torsional rigidity.

Although these "inspection findings" could be labelled as **LTB** or otherwise, but if viewing from a different angle, they might be better classified as some form of **acquired** "geometric imperfection" rather than the aftermath of "post-buckling." Yet not many among us or hardly anyone stood out to categorize these deficiencies as being "lateral-torsionally buckled" as yet, because for unsymmetrical sectioned members, the definition or the look of "being lateral-torsionally buckled" under this mode is vague for lack of "officially documented" mathematical or lab test proofs – then again not until proven, *something we don't see or talk about doesn't mean it doesn't exist*.

3.8 If Not LTB then Should It Be Imperfection?

Structural deficiency comes in various traits, which may entice very little interest to some or else trigger painstaking reaction to many others. By the plain sight of loose/sheared bolts, cracks in weld metal or base metal, etc., one might not have on-the-spot knowledge of the "how and why" but should have not only documented the deficiency but also should trace back to see *are these recursive findings*. And yet if lacking an exhaustive investigation – better supplemented with engineering-partaken confirmation – then it might be difficult to decode whether these "deficiencies with specific regard to their telltale deformations and/or damages" were the result of a single dominant source or from a multitude of many sources.

Structural distress of <u>diverse causes</u>, no matter displaying superficial or ostensible symptom at long last, usually unforeseeable up front but let it takes its tolls at its own pace − often by unveiling trifling trail at early stage before more severe consequence comes to light.

> These <u>diverse sources</u>, mostly obscured from the outset, could be planted in way back through Engineering and/or Detailing phase, Fabrication and Construction phase into the Operation and/or maintenance phase. Then from whichever as promulgated − as applicable – that might lead to <u>permanent deformation</u> *in the end*, otherwise could stay dormant indefinitely then germinate and flourish only when time is ripe
>
> On balance, bad breaks could "creep in" from many causes;
>
> > <u>To name a few:</u>
> >
> > *Global/local **3D** out-of-tolerance, loosened connections, overuse syndrome, bad detailing practice, acute environmental effect unaccounted for, normal (or abusive) wear and tear, rail misalignment, large deflection from elastic (or plastic) behavior, Pre-**LTB**, **LTB**, Post-**LTB** or some other modes of failure or even the consequences from <u>metal fatigue</u>, and most of all the <u>**engineering blunders**</u>*

As some of the **CRGs** exhibited <u>apparently</u> tilted- or warped-like symptoms that were far more noticeable contrasting those "inconspicuous hints" or "minuscule amount" of deformation, it really needs no special words to exemplify. In some cases in point from a global perspective, there were **permanent deformations** most discernible near the mid-span where there usually is the most obvious location to look for tell-tale "imperfection."

> *In that more often than not the top flange would lean laterally one way while the bottom flange would lean the other way, an indicative of suffering from excessive twisting and warping*

Deformations bordering the state of "Being Twisted/Warped" tend to show up more regularly to longer-spanned **CRG** or those of high **L/d** ratio that might occur either **temporarily** during shipping, transferring, unloading and/or erection stage or else **permanently** after any number of years in service. Although borne into unattractive looks as based on the appearance alone, fair or unfair so as being judged by the sight, yet most of these **CRGs** were held on functioning in the production line still. But there is a situation worthy of our precious while to look into more prudently:

> On active duty or not, nevertheless these structures with **permanent deformations** often hung in there with indiscernible metallic pain and discomfort the eyes might not see
>
> If so while lacking proper engineering-based investigation into what are the root causes and the true reason why the deformation has become permanent then, the distress of these girders, albeit toiling calmly as if pain free through service term, could be so deceiving to the discernments of many <u>unsuspected</u> Crain Operators, Maintenance Staff and/or the Plant Engineers, etc. Or in a way whether being seen or unseen by all, but attached to these girders there probably hung a hefty toll that grows and grows if the responsible party chooses to look the other way

Thus it is commonplace provided that there were no serious serviceability issues incurred, these formerly "wounded" structures would probably be kept up (1) for as long as deemed practical (or impractical as well) or (2) for as long as there is (net) profit logging into the book

Usually the show (must) goes on until the day that either these **CRGs** racked up too much maintenance burden (financially) or met their fateful failure (physically) beyond further repair (technically) yet as for the worst, some of the *base metal material at critical stress hot spots had gone through so much heating-reheating and cooling-burning from previous repairs that could no longer receive any further torturing from new welding*, period

Buckling or buckling-like events whether sparked locally, globally or system-wide has always been bad (and should be bad) curse to not only the **CRG** per se but also apply to structures of all ranks most of times. Should that unfortunately occur, it confers an impression of *engineering design fault* and the structure would be doomed in the public eyes as if *about to* fall down if not so already.

But what if the *condemned* **CRG** was still standing and had not been fated into more serious distress, and there is no indication of local yielding episode witnessed by visual measures, then how would anyone label what the **CRG** had been through when judging by these "somewhat <u>permanent</u> deformation?"

Seemingly a practical term is in need here to distinguish it from the aftermath of a genuine **Euler** buckling or a regular **LTB** event demeaning symmetrical **I**-shaped beams, but from an unofficial viewpoint let's say they have suffered a "**Technical LTB**"

To whom that may have concerns, is **Technical LTB** really that scary if not catastrophic? Probably not, but to some extent in a positive sense, these "somewhat permanently deformed" **CRGs** may not be as bad as envisioned provided that the structural member:

- Shows no sign of local buckling, has not fallen off and is properly supported at both ends
- Is torsionally stiff − debatable
- Is not stressed beyond yield − debatable, and
- Presents no "serious" serviceability issues, etc.

Whatever that is with respect to cause of permanent deformation, it is easy to hand in an alibi for it instead of trying hard for the fact, but still there seems to be some engineering-flavored flaw hidden somewhere uncovered yet, so no excuse if you please. Practitioners for sure would wish that the Modern Code Committees could appreciate (or commiserate with) the need into adding a special Chapter or otherwise a few paragraphs in this regard so that all ranks could be benefited and be guided on course for **CRG** applications.

Yet other than keeping our hopes thriving in the short haul or as has been in the past, providing ample lateral supports may be the only reliable strategy in averting **LTB** or **Technical LTB** in general. But:

Such provision would work out only if installation of strategically located lateral supports is a realistic byway and be physically feasible, too, or else count that as blessing if one could always put it through practice in those hand-me-down projects. Otherwise not so fortunately that, problems would still remain unsolved for so many cases in so many older Mills, for which installing lateral supports are flat out impractical and most of times impossible due to severe interference with critical non-structural utilities or unavoidable project constraints especially in those non-stop running Mills, then what?

How about the idea adding bracing in new facilities?

No matter how thoughtfully was meant to brace against lateral swaying movements of a main member or however prudently its section profile was furnished up into a boxed-like shape with lacings, struts, trusses or (superficially) rigid framing or some sort, but even with all that fancy setup *there is absolutely no practical means to prevent or mitigate the Z-rotation owing to the "acquired torsion" or its influence to the design of any **CRG***, be it of symmetrical sectioned or not, except only to reduce the damage due to torsional effects to an *artificial* minimum, or maybe not at all. Why?

> Because the installed universal position of crane rail has already implied/imposed where exactly the loads are coming from, and quite likely for a fact the **3D** applied loads or the load resultants do not pass through the cross section's **Shear Center**; then on the outcome from any structural detailing fixes, at best it may be theoretically **OK** for a singularly applied **Y**-load on symmetrical sectioned **CRG** but will never work out practically for **X**-load in any case

See the **Shear Center-triggered** trouble in the brewing once again?

Therefore on the flipside of **LTB** or **Technical LTB** prior to any (global or local) buckling to takes place, all Practitioners should take it seriously that:

> *There is a **much bigger problem** looming over **CRG** from **torsion** and **metal fatigue** that must be taken care of properly in all design*

3.9 What Could Practitioners Do About Unsymmetrical Sections?

Some of us are wondering still: Isn't there a resolution to this same old question (see **Chapter 1**) being posted here once again?

> From a normal but nontrivial insistence, all Practitioners were longing for is way beyond a *straight* yes or no versus *doing it or not doing it* but a more generic strategy and more practical methodology applicable to **Crane Runway Girders** whether there is **LTB** or no **LTB**. What works if it does should cover both the unborn ones not yet fabricated and those aging ones in need of maintenance repairs/upgrades − whether of **I**-sectioned or **unsymmetrical sectioned** − not the recaps based on outdated and mostly sloppy science

There's no need to go far these days for varieties of free engineering advice. There we have so much *oldie-but-sometimes-not-necessarily-goodie* structural design information or *half-right-half-wrong* type of **CRG**-related technical design guides or simply hearsays that are aplenty. Thanks to the widely available Internet access through button clicking or keypad swiping that most "Users" think they could resolve almost anything firsthand through **DIY** − beyond visiting physical libraries as in the old days – and be done with, but does that really help?

Upon fresh online literature search on **Crane Runway Girders**, there were no short supplies of purported guidance or opinion fitting various interests. Yet, most hardly went beyond general interests − just beware of some of the "not so hands-on information" appeared over and over on the Internet – that one should be very careful about what's out there and don't just fall for it with open arms right away, including the **Chapter Series** presented herein.

> Although it's a separate issue on discerning whether these information were more specific for refurbishing **older** girders already in despair or as words of advice on **new** girders yet to make a debut, or as generic tipping on girders of all ages and species, or whatever interest that these information were for, etc., but how adaptable these info were within any user's specific discretion would depend on:

- What resources were readily available, not after, but at time of need and

- Whether if the intent as "advertised" in these resources were truly applicable to solving our problems on hand

Or more accurately speaking, the truly "useful information" should be **CRG-specific** and **task-specific**; herein how beneficial that is for dealing with *unsymmetrical sectioned girders* is the key. Before buying into any as promoted, be thorough to identify and watch out for hidden misinformation or see if the materials were indeed helpful and not that packaged as submittal for pure academic objectives only. Finally, be on the lookout for those lackluster varieties that may misguide some if not all users.

Then from among those ever popular fixes, tips, guidance, opinions, equations, solution formulas, testing data and even spreadsheet-smart kind of design examples, etc. gathered from literature search, whether publicized or kept (fully or partially) proprietary, quite likely that any Engineer who has perused these technical literatures could provide an honest answer to these questions:

> *It doesn't matter if it's meant for Crane Runway Girders or not, but how much information out there were given with focus on **non-I-shaped** members? How many examples had dealt with **torsion** or even mentioned the "**shear center**" phrase? How many had given detailed advice on **metal fatigue**?*

What's missing as of this writing was the industrial-strength **CRG-specific** guidance. Any "Modern-day Design Guides" being honored as "Design Guides" should be all-purpose in principle if not all-inclusive but not some disguise behind a few bullet points, fancy sketches, screen shots or presentation slides for the showrooms or classrooms, etc. Frankly and earnestly it's worth repeating our sincere pleading here again:

> To all aspects of Structural Engineering: What Practitioners truly deserve were the **genuine** hands-on material with guidance in (1) helping out with important tips to follow and (2) pointing out hidden traps to avoid, etc.
>
> For certain there were technical savvy advancements much further than what were available several decades ago. Albeit that's good in general yet not an understatement articulated herein, it's proven, most were incomplete or piecemeal and even some of that were not very user-friendly for practical **CRG Engineering** consumption
>
> One would do better with information (1) for Real-Life **Non-I-shaped** Unsymmetrical Sectioned Crane Runway Girders − instead of loose coverage in the same way as old folktales snitched over and over on **Exclusively-I-shaped** girders, or some non-essential treats straight from beating around familiar Code formulas limited to **I-shaped** members or that more so with behavior conforming to **simple bending** and (2) the up-to-dated **R&D** advancements − unfortunately which in some ways, in rare scarcity if accessible, were for other non-engineering-design purposes

Despite what was out there or not there, Practitioners need some hurry-up offensive/defensive game plan that could help scoring some quick points.

3.10 Playing Safe – A Better Defense

One wonders why so urgent the calling for industrial-strength **CRG-specific** design guidance right now?

What are we − or more appropriately what are the girders − fighting against eventually? Besides material yielding and stability issues, in one catchphrase: Metal fatigue.

> Again, why the rush? Just pay a visit to some of the **older facilities** (actually not necessarily being that old in some cases); it doesn't take much to uncover from not afar but look closely within our Engineering/Inspector Line of Sight Perception. Deficiencies owing to metal fatigue such as cracks in the weldment, base metal or that at bolt connections could be more obvious than those otherwise hidden out of sight behind some unreachable obstructions – *some we don't see*

Defect of certain traits we don't see at a given moment doesn't mean it wasn't there or wouldn't be there. There is a timing factor to be recognized. For talking purpose − some of the deficiencies associated with bolting might take up to 60 years to develop but that with welding might surface within 10 or 20 years, or so sooner or later but give and take

Bottom line: *What takes to properly design against metal fatigue is in earnest need*

There are many Mills in active running with numerous (actually of *miles in length* in some Facilities) girders in hyper state – more like metal fatigue-challenged, that is. And many of them had been or would be in grave troubles in need of serious attentions much sooner than it appeared

And besides, it is never amusing to say that quite a few of these girders were not even of irregular section geometry but duly symmetrical **I-shaped** or *those with a cap channel on top*. Admit it or not, some of them were not adequately designed from the onset as if conceived with birth defects

<u>What makes it worse is</u>:

A lot of the sicken ones cannot be easily accessed for in-depth/proper inspection and/or investigation due to interference let alone coordinating resources for proper remedy. All that plus the fact, providing effective fixes to upgrade and/or making repair to existing **CRG** in the actively <u>non-stop running</u> Mills is always much more difficult than cultivating fresh new ones for an idled facility or those not yet existing Facilities − how much harder? Experience sure will tell

Ironically, all in respective proportions, whatever technical attentions and fixes these aging **CRGs** in these Mills need in this modern era could rival or be two of a kind with what the clinical attentions our aging population received in our human society – *after all, the applicability of the term "aging" or "old" whether to a person or to a runway girder is only relative*

Imagine the situation being caught up head on with these said issues, say, being technically unsuspecting yet assigned to deal with an ailing good old unsymmetrical-sectioned **CRG**, how would that feel like?

Wouldn't sometimes the situation be akin to taking a fresh lesson on structural rescue mission with no time to spare yet already deep in the course of a firefighting-like repair/upgrade engineering/reengineering project? What then in such a crunch time besides doing the *same old repair in kind* just like the *same old bygone* that so many others do? Or do we want to do it differently and do it right this time?

Do it right, yes seriously, but some of the not-so-confident among us may toil a mixed feeling: As if there were little or no clear-cut tips on when/where to veer one way or the other, neither were there reliable telling of subtle disparity between what should have been done and whatnot

At that point, how (not) miserable may one feel from the experience in handling both **torsion and fatigue** design mandates would depend on how profound the **awareness** and at what **skill** level one had in handling diverse issues innate in the fixing of existing unsymmetrical-sectioned **CRGs**

Clearly, so think and rethink again then ask; were all those well-established well-recognized "Official" Design Rules and Guidelines <u>adequately</u> (if not fully) vetted for treating **CRGs** of *unsymmetrical section*?

Certain "Official/Unofficial Rules or Guidelines" could have been traded up/down to becoming our "handy hardy shortcuts" fittingly as what were called for a while back (see beginning of **Chapter 2**.) But in the areas of minding both *torsion and metal fatigue businesses* exclusively of **CRG** interest, there just aren't many tricks of the trade in the commercialized tools for Practitioners to go by except by playing the good old scheme (engineering rip-off) of "**Being Conservative**" again and again, sometimes much like hedging a bet on the life of the structure yet not without its nemesis.

But only if staged properly and readily justified then, playing safe − with no hidden engineering scams − in normal process is a much better defense against unconditional hits and misses, numerical whoops, or those so-called technical inadvertences, and the like, but watch out and be prepared, it could turn out to be a process easier said than done.

The fact concerning **CRG** in general, the buzzword "Conservative" associated with playing safe often was unqualified, overused or misused in many engineering endeavors lacking explicit user discretions more than ever *in the areas of (1)* ***analysis*** *of structures with unsymmetrical section profile and (2)* ***design*** *of which involving* <u>*torsion*</u> *and (3)* ***qualification*** *of structural adequacy against* <u>*metal fatigue*</u>, etc.

Quite a few of us − some of the overconfident but misled or misinformed yet so self-assured Practitioners − were not properly equipped to justify prior to making rhetoric claims or documented statements by simple phrases such as "My design is conservative therefore it's **OK**" or "The stresses are low so it's adequate," etc. especially in the court of law when our opponents don't fall for such account or flat out disagree with our self-acclaimed stance of being too moderate, too conservative or otherwise.

On occasions through experience, even making a simplest engineering/design assumption could become questionable in engineering-fashioned legal sense; for examples:

- Why making such assumptions?
- Is that the way to qualify structure against metal fatigue?
- Why was that a simple support instead of a fixed support?
- What is the fatigue strength at this bolt hole?
- Why should the shear center be here? …

Notice that some of the questions deserve a fairly lengthy answer.

> When any of the inquiries like those were handed down to us and if we failed in backing up for the "legal case" with accredited engineered numbers then chances are it would invite unfavorable cynicism from peer reviewers or arbitrators (plaintiffs or defendants alike) and could further us to nowhere but reinforcing opponents' skepticism. Whatever the outcome from the as-said testimony would depend on peers' roles and their positions − for or against us − since they could either act as if our friendly associates with great helps or as foes pulling our legs during a legal flip-flop.

One of the much bigger Engineering-flavored social (or Social-engineering) tribulations, being much more annoying than many other issues, could be dramatized somewhat as follows:

> Some so and so − entrusted with a technical <u>mentor</u> or project <u>leadership</u> role − would lead in the "pseudo conservative" uproar over a certain contentious subject in focus but on <u>dubious grounds</u>; and then other accomplice(s) perhaps of <u>inferior rank and file status</u> would blindly oblige to a matching controvert possibly campaigning even louder whilst making a "technically or politically" correct effort but all devoid of authenticating if the basis of advocate is relevant or appropriate. And through and through, similar drama went on and on, project after project …
>
> To finish with the saga, after many years or decades of being "technically" inappropriate, incomplete, incompetent or incorrect, "they" let the "bad science" takes hold and overshadows the real problem indeed. Then it went on over and over again – *the typical retort to questions that some of these "amateurs in disguise" give is pick and choose to answer the easier ones then avoid (dodge) answering the unanswerable ones or denying there is problem instead of facing it.*
>
> Sounds too familiar in real life but found in too many Engineering functions and positions, isn't it?

Among numerous bona fide dilemmas sometimes there seemed no one realizes or is willing to concur with the existence of some particular Engineering-social problem(s) until a simple but indisputable **CRG** branded question was raised by some daring newcomer(s) to the (already-professionally-aged) guru out of the blue, saying:

> *If the design of your/our **CRG** was truly conservative then how did the weld crack right here in the compression flange, or why had the bolt sheared or missing over there?* Or asking, *do you really think there is no torsion?*
>
> It would have been OK only if in sincerity making a confession/concession; other than given an "I don't know," or "That's the way it is" from the responsible guru, got any credible answer(s) yet? What could be worse is that sometimes the guru didn't stay in the loop much longer to witness the cracked weld or missing bolts or something worse

After all, we are facing **CRG** of unsymmetrical section with a lot of official unknowns and/or as much of unofficial knowns, or vice versa depending on how one would judge this very special engineering reality on solid grounds or on hollow grounds.

> Being an unsuspecting Practitioner, one may happily approve/accept that doing everything the **I**-shaped way and with no hindrance from trusting **Flexural Analogy** is the way to go and with very little defying, confirming, complaining or questioning, etc. But was it truly **conservative** or was it really the proper way at all?
>
> Most of times it is much easier to justify being **conservative** (with conditions) for **I**-shaped members designed under limited or with no binding directive or design mandate against metal fatigue. But for unsymmetrical sectioned **CRG**s' sake, it looks like we were left technically desolated or basically "on our own" as we pointed out earlier, at least for the time being
>
> *Just remember this **Flexure Analogy** does not work for unsymmetrical sections for many reasons; even if one tries, then what is the free-body force diagram look like?*

And yet whether doing everything on our own or working in a teamwork environment, we should always remind ourselves to fulfill engineering obligation with *long-term functionality and structural safety* in mind. As for better engineering treatment of those exceptional **CRG** species, until the abundance of truly useful information on "what to do" were on hand, it might be prudent to start from drawing <u>unlimited intelligence</u> on "a lot of what to avoid or not to do" whereby in the interim knowing what to avoid or what not to do sure makes better sense.

3.11 Avoiding Unsymmetrical Sections

There are times we need a quick and decent prompt to disengage with section profiles having odd-shaped geometry; <u>only if we could</u>, otherwise the better (if not best) approach is to follow the first and foremost "What-Not-To-Do" as strongly "hinted" in the **AISC Commentary**:

> <u>*Don't use unsymmetrical section*</u>
>
> > That meant well indeed; and in fact regarding our preference in **CRG**'s configuration, must we single out the most favorable and most important element outshining all else out of a collective Not-To-Do list, that being recommended should be it then
>
> Avoid unsymmetrical sections if we could, of course when given a choice; however, it's also good to know that "avoiding unsymmetrical section" is not necessarily the universal be-all-end-all remedy applicable to all practical situations

"Avoiding unsymmetrical section" is more hands-on for newly designed yet to be constructed girders; that same idea may not be consistently obliging for those aged old hyper-up-and-oddly-profiled girders being repaired or upgraded and/or those *not quite ready to be replaced*. Why?

Most (if not all) Code(s) might have been slated with largely good intention and recommended the "do-not-use-unsymmetrical-sections caution" clearly, but inadvertently kept lenient on some of the stricken real life circumstances

For instance, *whom could we turn to* for guidance when taking it for the situation **what if** we were preassigned/prearranged to retrofit **CRGs** having unsymmetrical sections high and dry meanwhile trying the best to cope with Facility Owners' demand of keeping their good old Mills up and running − not only shooting for the turnaround to be as soon and as cheap as possible but also for results to be as good as wished for − under projected budget and meeting "our" obligation?

"Avoiding unsymmetrical section" is a realistic start destined for technical simplicity, so unless if ever there were added options or other ways around then perhaps all Structural Engineers would love to avoid dealing with unsymmetrical-sectioned girders in the first place. But no matter how logical or how great the "do not use" advice may seem, from which what concerns the most should be the "Retrofit Sectors" especially those with no better choice but already doomed to hang out in those existing Mills and had to deal with unsymmetrical sections fairly and squarely.

Envisage being "technically" smart for "analytical pleasure" if in some way "financially" silly, and how hurtful "emotionally" would that be for the entire Industry Worldwide to command a full-blown universal ditching of all those classic yet humble unsymmetrical **CRG** and replace them with all brand new symmetrical sectioned members in each and every *legacy facility*?

Some realism we could all relate to:

Wouldn't that be much like etching a crafty mental pledge to always shell out for brand new vehicles whenever there is need in repair of our (energy guzzling) old clunkers? There is a serious business side to this: Where is the (financial) resource? Facility Owners' minds are not that engineering oriented but rather more financial oriented comes to dollars and sense. They would not be convinced easily unless the net gain from girder replacement is worthy of the cost (headache) of re-engineering

The bottom line, again, "avoiding unsymmetrical section" may not be that easy to go along with across the board. What's important to us is not only should we *read the lines* outwardly by the literal but also we should look into the situation and *read between the lines*.

3.12 Local Buckling – From Gross Section to Effective Section

If indeed unsymmetrical-sectioned **CRGs** were unavoidable then, the next best strategy would fall on the prevention of "local buckling" to the best we could.

Concentrating exclusively on longitudinal stress for discussion as in the next part:

The truth maybe, not all of us were enthralled in the hardcore heartfelt theory behind local buckling during practice. Even if some of us do but not many were absorbed enough in what may come after such momentous adversity took place in real life, albeit most of us do duly retain enough know-hows so to carry on by the Book as one of the upfront design defense strategies − certainly a good and farsighted habit to form − in avoiding the fateful buckling event to come about. Nonetheless if inspired, Readers are encouraged to pursue further.

As to recapping what structural stability issues are about in general: A structural member's or some of its components' vulnerability to buckling occurrence has everything to do with the **effective slenderness ratio**. And besides that:

(a) On evaluating **global** stability of a column/strut under axial compression applied from end to end, the familiar **slenderness ratio** was normalized as **L / r**. Whereas in practice the normalized ratio − with **r** being the least value domain-wise − was modified with an **effective length factor** into (**k * L / r**) becoming the **effective slenderness ratio** in that the **k** parameter would depend on the member's end restraint situation, material properties and a few other "idealized" parameters and other well recognized assumptions maintained per **R&D** initiative

(b) Aside from concerning **global** stability of an entire member but for **localized** buckling of **local** cross section's component elements − such as an outstanding flange, the protruding portion of an attachment or a web within the confine of cross section, etc. − by and large the individual element's aspect ratio, particularly the thickness **t**, would chime in as one of the controlling parameters

The **modified** governing **aspect ratio** as summarized per (latest) **AISC** could take after the form per one of these expressions: $\mathbf{k\,(b\,/\,t)^{n}}$, $\mathbf{k\,(d\,/\,t)^{n}}$ or $\mathbf{k\,(h\,/\,t)^{n}}$ whichever designation **b** (breadth,) **d** (depth) or **h** (length or height) is appropriate, where once again **k** has something to do with the element end conditions, material properties, unique feature of how the section was built-up and the stress category of interest, etc. while the quotient inside the parentheses is simply the "element aspect ratio" and normally the exponent for compressive stress **n = 1** and for shear stress **n = 2**

When developing a cross section from as many component elements as practical, the best line of defense against local buckling for **CRG** (or for any structural members) is to maintain the aspect ratio (width-to-thickness or height-to-thickness) of each component element below the Code recommended non-compact and/or slenderness limits.

Doing so the component of interest should merit **R&D**/Code's blessing thus would be less prone to local buckling when subject to compression and/or shearing (notice that both states could be expected from flexure and torsion)

For the gross section as to resisting compression as a whole, what if a certain element's aspect ratio had exceeded the Code-blessed threshold value?

Depending on the magnitude of stress and the level of vulnerability to failure while under axial compression, the element may maintain its gross entity intact and stay as is or otherwise one or more nodes or segments of certain element(s) may show sign(s) of bulging out of plane or being bent out-of-shape, which could be a precursor to local buckling or as its aftermath. Further from that, there could be two possibilities: It can be (1) as severe as triggering a chained event into a total (system) collapse or (2) as mild as losing effectiveness in resisting compression

Other than a total collapse following a local buckling event, as long as the element is still standing and at least staying "in one piece" then what brought about is lessening the element's gross section into an effective section against compression

Ideally for the gross section in focus,

When all is well in consigning the aspect ratio of each and every component element under the respective slenderness limit, the entire cross section would be "fully effective" against local buckling under compression and/or shear

Fully effective implies the **Flexure- Related** cross sectional properties for flexural stress evaluation could be based on the as-given and as-preserved gross section joining all elements as fabricated as-is with each of that contributing its full span of **b**, **d** or **h**. Given such expediency (we'll see the otherwise) all thanks to a compact section being in the clear from local-buckling in view of compression (and/or shear) to start with, or we could safely say that the cross section qualifies to be local buckling free

When the usage of gross section-based **Flexure Related Properties** in practice is wholly justified, the calculation of flexure-induced stress becomes a much more pleasant process. However but not so fast though, whatever the hard-earned contentment from dealing exclusively with gross section merely minted on the better side of the coin. There's no need to explain much – the flip side of the coin is how to properly deal with effective section.

A basic understanding of difference between gross section and effective section is only the beginning; it sure doesn't mean that we are entirely off the hooks **technically** because we have not probed further into (1) the loading side and (2) the associated data processing side yet.

3.13 Loads and Load Combinations – Data Processing Woes

As so well interpreted all along, to master the **Total Engineering** of **CRG** meeting all-inclusive design intent takes much more than the faint-of-heart attitude; yet on principle still the processing flow for this rather involved undertaking is not much different from solving most structural engineering problems.

> Herein at completion of the process, what matters the most to us is in the details revealing (1) what has been done and (2) how it's done during the process and (3) to what extent

Qualitatively speaking from a reviewer's takes on the assessment of design at its closing stage, if that were appraised merely by the gross amount of effort and/or resource expended (including that *wasted* or *misused*) then it's not enough proof of true achievement.

> Simply that does not count unless the quality of the deliverables was attested (1) through unconcealed demonstration of how everything was fixed, and as applicable (2) through provision of justified reasons on why certain measures during the proceeding were *omitted, undeveloped, not carried out completely or not at all* among other evaluation criteria

Tweaking for a better or ideal structural configuration tailored to the imparted specification often takes up multiple rounds of trial-and-error attempts involving incremental optimization over the progressively as-estimated, as-given and/or as-modified cross section geometry; such idealistic goal might not be achievable as always but at least it is a normal objective in most **CRG Engineering and/or Reengineering Missions**.

> As far as data handlings are concerned, some of the indispensable errands emerging throughout the progression of **CRG** design improvement from start to final attainment involve succeeding series of information compiling, evolving, tracking, backtracking and consolidating, etc.
>
> In all **CRG** applications, once upon completion of initial load analysis, it is time to wade into the vastly unbounded numerical infield to straighten/sort things out ready for the next leg of journey and then many legs there after
>
> Easier said without doing the real thing for what intent as said; it always sounds too straightforward at face value; yet with ultimate design goal towards meeting *fatigue design mandate* in tow, it beckons a reconditioned mindset and renewed approach beyond old-schooled practice routine; because implicitly the quality of data process can take hold and dictate the final engineering result very big time, there would be traps hidden near and far in every aspect from enveloping of the all-inclusive load response data to the compilation of entities' numerical maxima and minima, etc.

Once again, fatigue or no fatigue, keep in mind a very unique **CRG** loading feature; it's applied at the rail top off **shear center**. The big deal of that is every individual load **P** comes with its own version of **delta**. The bigger deal is to watch out **P** may or may not switch sign meanwhile certain **delta** could switch sign, too; clearly torsion comes in very turn and there is no way out

By all that, suddenly an engineering problem becomes a database problem. The success of such mission has to rely on a well-strategized **data processing scheme** as solid base – way beyond relying on regularized simple-bending-fashioned sights and senses alone − *in a way we are banking on balanced data collection to sort things out and unbiased information digestion all for nothing else but on engineering purposes*

In the end all works would need to come to our own satisfaction first before being reviewed by others.

A reviewer should for the least *look/read between the lines* into areas regarding (1) how methodical the analyses were carried out as to the broadness of applicable loads and effects – *handling of multi-wheeled moving loads for instance* and (2) how the structural adequacy was justified with regard to all-inclusiveness of all credible load effects − *handling of various **P-δ** effects*, etc. Therefore there is no sense spending/wasting time into too much detail if already dissatisfied with treatment to those big-picture items

Reviewing into the details:

Take the **nodal longitudinal stress** (or with similar idea for shear stress) for example:

Basically, across any cross section of any typical **CRG** member under intended loads, from which any constituent element (a flange or a web) whether in full or in part could simply be in one of the two circumstances for any load instance:

- In **tension**, as result from selective set(s) of single loading event or that following a specific set of load combination or

- In **compression**, yet it could be under the same selective load sets (as that for tension) and/or completely different sets of loading event or load combination; *or in some cases the element may never see compression at all, and vice versa*

Before calling it all good to go near the end, the closing of **CRG** qualification process requires *detailed **node-by-node** fatigue strength assessment*, which depends on the extreme positive and negative values retrieved from a depository of nodal stress' peak and valley that quantitatively fluctuates with how the associated loads were combined; that is the reason why keeping tap of the sign each number may carry and how that may change around throughout the process becomes utterly important

To make the point across on how easily it is to fall off from our standard expectation with focusing on the longitudinal (or shear) stress of a "certain **z**-strand" within certain **X/Y/Z** nodal confine:

As the analytical work continues well into the load combination stage, numerical turmoil seems to flood over our desktop/screen. In interest of *fatigue strength assessment*, what enumerated per our natural intellect could be veiled into an opposite or rather unpredictable ending that could be way out of line with what as estimated once the tracking of reversal of *"load orientation or stresses"* went into play

> The most common issue (or non-issue): Tension can became compression (or plus became minus) or otherwise unexpectedly, for example, *seeing* astonishingly *certain node in the compression flange suffered from tensile fluctuation* wondering how can that be possible and why it happened

It is not uncommon being taken in at seeing what seemed logical under "our normal guesses" per our "**non-CRG mentality**" to turn out not as anticipated – for instance, take certain internal stress vector(s) for a certain "longitudinal **z**-fiber/strand" (1) which should have been in tension or in compression or (2) that a fixed numerical value should have carried either a positive or negative sign, etc. – but then numerical sign change is only a minor thing, the much bigger surprise is some of the modestly stressed **X/Y/Z** nodes although passed the non-fatigue assessment but could fail by **shear fatigue** very badly, and the worst of all is *not knowing what fails if not revealed through properly prepared calculation*

The different behavior in what we are talking about further from *simple bending* is in the longitudinal (shear) stress resultant, which to all **CRGs** is a mix breed from flexure and torsion. Provided lacking (1) a sensible prompt proffered through robust data administration-oriented means or (2) some hardy-programming geared for fatigue strength assessment purpose, there it won't be that easy to tidy up the aftermath of load combination nicely or correctly

> Then very likely our cerebral-based "hit/miss or right/wrong" conjecture could flip in/out in midst of a specific phase (or any phase) of calculation. Easily, things could turn out far off from our normal control because "every **CRG** application" is different – in terms of loads and/or configuration for the least − to which the provisional statistics from each and every analytical session could seem so "uniquely illogical" in its own way that hardly lend out specific cognitive intelligence to pin on
>
> Thus from application to application, the engineering process flow may seem to follow monotonous trails but the uniqueness of profile geometry and the detailing features as dictated in each **CRG** numerical problem-solving experience gave very little imitable pointer to draw on for benefit of engineering of next-in-line applications – technically *can't just bluntly copy and paste or risk of garbage-in-garbage-out situation to show up*
>
> Why so? *To find out the practical way just try a hand on several **CRG** applications, for each take it from tracking the combination of flexure and torsional effects owing to reversible loads and mixing it all up with fatigue strength assessment for a proof*

Regardless to the numerous **CRG** facts agreeable or not to all Readers at this point, there were dissimilarities in numerical attributes – *reversible or irreversible load sense, ±δ and the associated upper bound/lower bound load response magnitude(s)* − defined among various load terms, whereas being standalone as an undressed entity, an individual load term would generally take after a unique symbolic meaning (**D** for dead load, **L** or live load, or **W** for wind load, etc.) usually with fairly limited numerical noise to be heard from when playing each one by itself individually.

What made sense hereinafter is not much different from what made sense in most applications.

In a full-blown **CRG** application as to meeting the requirement stipulated in the given design spec, each individual load term would take its own turn and be queued in position (*popped* out of data depository) for a call of duty. All loads are of equal rank initially, by which no one load is more significant than the other until appearing on the numerical processing stage on cue from a specific load combination equation and then by a governing formulation adapting to specific type(s) of stress.

Under normal practice, the bare bone analytical process starts typically right after applying the specified load to the structural model:

> A little more detailed insight:
>
> Soon as a loading event (load term) was called upon joining other loading event(s) as fellow operand so then both are related to a specific load combination equation, each load term would be tagged on with a load factor as numerical modifier prior to being added up into a final sum
>
> To each **X/Y/Z** node:

It is clear that each operand for the specific node had brought with its own (plus or minus) sign prior to joining in the combination act; what churns thing up in a somewhat premeditated manner is the “not so intuitive” load factors (such as 1.25 or 0.5, etc.,) which is pre-assigned, per Code or design spec, to each and every unsuspecting load term – think about how 1.25 to one type of load may make a positive value and 0.5 applied to a different load type may turn the final sum into being negative

What to take notice from is the dynamic nature of the result comes after the load combination process; what appeared ordinary and predictable in the beginning could become abnormal and unpredictable, or, unless all load factors = 1, but is that really true?

Yes and No, sometimes not until gone deep into the heart of **CRG** application, one may or may not fully appreciate how an implementation of “simple” engineering procedure would have to call for a heavy-duty database management setup to deal with the “mess” even if the load factor equals to “one” or whatever

Let’s say under the simplest combination case such as “dead load **D**” plus “live load **L**”: **D** + **L**,

In which although there is “generic” plus sign preceding the **L** term (or any other load) but in actual application, we need to distinguish whether **L** load could act in reverse sense or not – because some loads do and some don’t − if it’s true then we must play the combination game twice and store the result into two separate data depositories: One from **D** + **L** and another from **D** − **L**

Nothing fancy there as pointed out but one must think it through further, *wouldn’t it be twice for lateral load but only once for gravity load in this case, or even more?* Or we might as well treating **D** + **L** and **D** − **L** as two different cases or that as we all do as for **non-CRG** structures?

Simple as that but the fact is, things can get much more complicated sometimes; one can try the hand at the framing of a *simple* residential structure’s gable roof truss framing under the influence combining “Dead Load **D**” + “Snow Load **L**” + “Wind Load **W**” for an inspiration:

Through different combinations of unevenly distributed snow with variations of wind pressure on various portions of the roof, some may find it incredulous that there could be 70 plus load cases derived out of a modest (**D + L + W**) formula. For that doing straight engineering analysis on behalf of **CRGs** is no big deal but the tricky part is in how to “systematize” the result for fatigue assessment which does not apply to qualification of residential roof framing design

Example like that is one of the murky got-cha situations that some of **non-CRG-oriented** Engineering Experts failed to pay attention to how numerical data were to be stored, modified or retrieved in the fatigue assessment routine; for which if we were technically shorthanded in facilitating an “effective” data processing means so as needed, often we turn to taking shortcuts carelessly like either doing the fatigue assessment half way or else making controversial engineering judgement or pure guessing. The subtle message here is *“don’t play smart by guessing or pressing the design outcome.”*

To make easy in the ensuing “**fatigue strength**” assessment process, almost anything or everything numerical-driven needs to be nurtured into as many normalized numerical domain(s) as practical and stored (linked) under respective rank(s) of hierarchy, otherwise watch out and we shall see what and why in upcoming **Chapters.**

Here are some other got-cha situations but of different twist:

At any **X/Y** node within a cross section, there are many (hidden?) factors that can "transform" the stress pattern from one state to another (for instance, compression or tension):

(a) One may need to take note of how conditions change in general; if certain load combination formulas involve reversible loads (lateral thrust load **Px**) then what noted as follow might be of interest:

- At times, the torsional stress reversal effect at certain **X/Y** node could be too prominent under a particular set/type of load (or load effect) thereby the structure − even for symmetrical sectioned member − could have behaved (not) so predictably or rather (not so) erratically depending on (1) where the **elastic centroid** and where the **shear center** were located and (2) the relative distance between the two nuclei – *much more unpredictable if too far apart*, that is

- The intensity of certain brand of stress in selected hot (cold) spots of certain cross section (i.e. say, mid-web at the supports or bottom flange near mid-span) may be more (or less) sensitive to reversible effects prompting sign changes (say from positive to negative) or *situations that might lead to fatigue failure* following a specific set of load combination(s) contingent on the *node-specific* fatigue strength threshold value

- At all times, one needs to keep in mind **CRG** is always under assaults simultaneously from flexure and torsion because of **P** and **P-delta**. Thus pinpointing the whereabouts of hot spots involving stress fluctuation − especially **for unsymmetrical sectioned girders** subject to so many "variations" of load effects − involving ±**P** and ±**delta** could only be impossible (or a wild guessing game) if we do try without detailed *calculation*

  Let's say by non-fatigue-adapted perception we have already located a few (obvious) stress hot (or cold) spots, accordingly whereas barely a couple inches away from these hot spots, for which we anticipate the stress value would not change much or change sign by (normal) guessing, right?

  Yet, provided that certain **X/Y** nodal coordinates were correctly identified and matched on the detailing/shop drawing, the (dramatic) fact is that certain brand of stress at the cluster of those closely spaced nodes may or may never switch sign at all for non-fatigue load, yet the opposite could be true after a bona fide fatigue assessment

  Disparity and unpredictability in the state of stress between any node pair in close proximity could be entirely negligible or be very significant as either situation can be so dominated by (1) the loading attribute constituent and/or (2) the inherent statistical dynamics imbedded in the local section properties even from very mild physical variation in between (among) nodes

  > The factor of influence of that as mentioned could be as simple as the nodal distance to the **elastic centroidal axis/axes** or to the **shear center** − or be any sibling member property from either **Flexure Related Properties (FRP)** or **Torsion Related Properties (TRP)** group such that any load (reversible or not) in participation would have either no effect in reversing the sign at all or even if it does but could be in such random fashion beyond speculation; watch out carefully however, *the opposite maybe true, too*

- "Aspect ratio" takes command of a component's effectiveness against compressive or shear stress. Here is a further twist of fate in this regard:

  > One element may be fully effective for **X**-bending but partially ineffective for **Y**-bending. That is to say, not all nodal points could be treated equally in flexural bending stress (or shear stress) calculation

  See that some node(s) may associate with element(s) that were ineffective in resisting compression, so if not compression but what about its effectiveness in resisting shear? See the problem? It is not the "what" part that bothers but the "how" part does

(b) Always do the math and drill the numbers:

Instead of merely **speculating** the fatigue strength assessment results, one should always bestow equal **numerical** respect to all participating **numerical** attributes from both flexure and torsion on the same playing field and watch how (1) the section properties together with (2) the structural responses may hammer it out **numerically** when dealing with stress reversal (and deflection) issues.

Section properties such as *moment of inertia, principal axes, shear center and warping constant*, etc. that obviously dominate the atmosphere that **CRG** geometry breathes in and out on the global scene. Yet one should not overlook those other not so notorious entities in the local neighborhood, namely:

- The **X/Y** coordinate of the nodal point in a cross section with respect to the **elastic centroid**
- Section modulus, **S**
- Element thickness, **t**
- Flexure first moment, **Q**, etc.

After all, these are the local numerical partners and associates working behind the scene (i.e. through equations and formulas) that each one of which would be entitled to dictate in its own way what exactly the stress magnitude should be and, most of all, what sign it would end up carrying, certainly no wild guessing could surpass that

What being revealed so far were some of the critical orders to monitor during a methodical "engineering review process" whenever weighing on fatigue stress reversal effect. If only we were serious in recognizing the importance of how each one entity can influence the design then we shouldn't brush this issue under the rugs by uttering an expression such as "It Is Conservative" without pinning the fact down by numbers.

Thinking that we had come to a good stopping point on local buckling and gross sections, but why should we be sidetracking into loadings and stresses as we just did? The fact is, bringing it up sooner just so we get the taste of how bad a situation could be during the stress-calculation ordeal and thus we would appreciate the "benefit" of using "gross section" geometry much more often or as much as we could practically, otherwise we may "pay" when time comes in dealing with "effective sections" later on.

3.14 Effective Section – Post Buckling Geometry

There were occasions in that retaining each and every component of the cross section at optimized aspect ratio becomes impractical or unfeasible. In other words, a certain element where its width-thickness ratio as given has exceeded the non-compact/slender *limits* − naturally these *limits* should be **R&D**/**AISC** blessed and recognized – then what? There is a tendency that a portion near the unsupported tip of that certain component might buckle locally from compression.

Next question: What if one of the elements had buckled (locally) due to compression?

To simulate the loss of longitudinal stress' resistance, the protective measure is to settle on a **partially** effective section by excluding the marginal portion(s) that has "buckled" away; that's that but after all, here is a follow-up from previous line of defense, more specifically:

If working with unsymmetrical sectioned member is inevitable then it is better still to avoid dealing with partially effective section by keeping the aspect ratio of all section elements under the non-compact/slender threshold. Otherwise it is necessary to calculate and maintain **two** separate sets/modules of **FRP**

- One suite/module to base on *gross* section
- The other to base on *effective* section

> Wouldn't the emergence of wicked "effective sections" make the data handling chore much more tedious? Yes, any portions or zones tagged as in a class susceptible to being post-buckled would be useless in resisting compressive stress (*with the assumption that shear buckling is precluded*.) Thus to all unsymmetrical-sectioned members, this is a precursor to all the data maintenance hassle that will follow

In essence, effective section could be regarded as the left over geometry (post-buckled) off the gross section. The water-downed leftover in either the flange(s) or the web is arrived at by removing a portion of some of the pertinent elements that was deemed ineffective after it has already (presumably) buckled due to compression − *but not yet for shear stress because rarely the ineffectiveness with respect to compression and that due to shear can co-appear in the same segment; and if that is true then not only the full segment may fail but also the entire cross section may go with it, too*.

As to the "intricacy" in keeping tab on or keeping up with various categories of stress owing to each and every load type and the stress resultants from various load combinations, there is a much complicated story behind; picture this:

> If the task(s) of identifying *effectiveness against compression* were carried out for only "one single **X/Y/Z** spot" then it could be a piece of cake; comparatively speaking though, but to perform a full-blown stress calculation fulfilling an all-inclusive evaluation at "multiple nodes dispersed all over the two-dimensional **XY** cross section and then onto the third-dimensional span-wide taking in all **Z** stations" with each **X/Y/Z** having unique detailing feature and to "**keep track**" of their numerical whereabouts in the data depository, it would be a totally different story

What then? We could imagine a situation of how might we handle it in technical sense:

> Imagine what comes at/after post-local-buckling:
>
> If any portion of the cross section having several thin/slender elements (each having a relatively high aspect ratio beyond the applicable slenderness limit) has been discarded from resisting compressive stress, then aren't we suppose to exclude it or to keep it in the active database confine when calculating shear stress?
>
> > Herein it's like pitting *flexure compression* against *flexure shear* for attention, isn't it?
>
> Don't answer just yet but press on; whether we make a point to take care of the issue or not, a data management-related problem is there to strike as soon as we start "**keeping track**" of things of this sort − no escape even for "symmetrical sections with stocky components" and we'll see why

Finally a few words of caution on effective sections that may affect the flange geometry of some but not all **CRGs**: There were minor (or major) differences in the limiting width-to-thickness ratio (the $\boldsymbol{\lambda_r}$ parameter) for rolled sections from that for built-up sections.

Should the latest **AISC Table B4** (as of this writing) be inferred as design update, refreshing, reviewing or re-qualification purpose then, it may be necessary that the compression flange of certain <u>existing</u> **built-up** girders be checked again to stay in current with the effect due to **further flange reduction** provided that the flange effectiveness were qualified per criteria given prior to the **AISC Black Book** intent.

3.15 The Tricky Effective Sections

Think it should become fairly straightforward once caught up with what does effective section mean – comprehended in geometric sense – but think again, especially that associated with unsymmetrical sections; all because of the hassle in the handling for unsymmetrical sectioned members in many ways were nothing but devious in actual application, or so being "tricky" in several ways.

Contrasting the expediency in applications with fully effective (gross) sections, here is the effective sections' **tricky part #1** regarding the so-called *effectiveness* in practice by the Books:

> Needless to stress further but say it once more; understandably for a specific element to be fully effective in absorbing imparted compressive stress to fullest extent, its aspect ratio must stay below the prescribed upper limit to rule out local buckling
>
>> To simulate a compression-triggered post-buckled element, the portions extending beyond the aspect ratio's upper limits are deemed ineffective (useless) and thus supposedly be "tossed out" but, hold it for a moment there; in this case even though the "chunks or pieces" that had been *algebraically* discarded but *physically* they should remain "intact" on condition that no parts had fallen off yet
>>
>> Thereby as long as the *shear plane remains smooth without exhibiting distortion or series of "wrinkles"* then, it should be reasonable to "assume" what discarded on behalf of compressive stress albeit were excluded from the gross section might still be and should be functional for "streaming along" the in-plane shear stress from element to element provided "shear buckling" does not take place. Why say so then? Or if not so, can anyone offer an official clarification, a better one?
>>
>> Because **typically** the threshold aspect ratio stimulating shear buckling is much higher than that attributable to compression – *therefore buckling under compression and buckling under shear might (or should) not happen at the same instant in the same element at the same spot*. In other words by that logic, it is not likely for a specific **X/Y** node of a certain cross section's element to experience shear buckling failure before seeing some isolated local buckling event already taken place at some locality **relatively remote** from that **X/Y** node all due to (excessive) compression. (*Interesting detail for Readers to find out the reason why*)
>
> That is to say, albeit one element may have been buckled (locally) in part as stimulated through axial compression yet technically it is still fully effective in transferring in-plane shear because the shear-flow is still streaming through everywhere provided that (1) *all elements were still physically connected and* (2) *none of the elements had been buckled involving shear*
>
> So finally, but not quite done yet, as per structure's response to **both** flexure and torsion influences, in-plane shear (stress/force) still flows throughout each and every component of cross section *provided* it meets the "applicable" element aspect ratio criterion per shear buckling. Or by that, one could interpret further as whatever **mishap** could happen from (in-plane) shear should always lag behind that from (axial) compression. Question:
>
>> *What happens if part of the cross section element is doomed to buckle in shear*?
>>
>> The bad news is, without using stiffeners, there is **no practical means** to construct an effective section in normal practice based on post-buckling due to shear let alone carrying on as if it is either/neither dependent on or/nor independent from the effective section on compressive stress' behalf
>
> Thus practically there is barely any good news in dealing with shear buckling and/or post-shear buckling events because the cross section in "normal" engineering view would be rated as being *failed completely*. But the only semi-good/bad news is that "normally" we must use "stiffeners" per latest **AISC Chapter G** (as of this writing) to aid in resisting shear in a way precluding the element from being buckled in shear, or else it no longer works no matter what
>
> But then the biggest problem with that is not on how to implement what stipulated in **AISC Chapter G** but on what affirmed in its first paragraph that goes something like this:

"… This chapter addresses webs of single or doubly symmetric members subject to shear in the plane of the web, single angles and HSS sections, and shear in the weak direction of single or doubly symmetric shape …"

CRG or **non-CRG**, what that really means to us is pretty clear that members having unsymmetrical section are out of it by the Book by default for one important thing, the stiffener weld would bring down the native strength against metal fatigue, or else we, the Practitioners, must make certain that shear buckling doesn't take place, period

In an all-inclusive **CRG** engineering's relevance, we are forced to deal with the unsymmetrical sections' paradox in a somewhat creative way, technically and numerically.

So even though a portion of certain elements were deemed ineffective, yet the gross section geometry in its entirety could still be counted on for **TRP** related calculation **except** for cross sections built up from unfavorable elements with (1) extremely high aspect ratio such as very wide and very thin thrust plates or (2) very long and very slim struts or (3) those lacing angles that are susceptible to shear failure (per fatigue and/or non-fatigue assessment criteria) in addition to buckling under axial compression

Notice that the word "**except**" is the key; but if so unfortunately and those unfavorable conditions were true then technically and practically "we (you) are really on our (your) own"

After all, one can "see" for practical reason, the definition of effective section of **CRG** importance is meaningful much more for compression's sake rather than for shear. However, even though the cross sections were built free from using very wide/very thin thrust plates or not very lengthy and not with very slim lacing angles, etc. the **FRP** for flexure-related stress must be **calculated twice, separately**:

(a) For flexural bending stress to base on the *effective* section
(b) For flexural shear stress to base on the *gross* section under the assumption that shear still flows through the cross section

There it goes easier said than done with the so-phrased "**calculated twice, separately**." It could be a perfect brewing into an unexpected data bookkeeping bottlenecks if not fully primed for taking on the ensuing mess and maintaining all raw data and derivatives in separate numerical depositories.

To get around troubles by being (ultra) conservative only when doable, one could always execute the **FRP** computation only "**once**" by using a common set of ineffective section geometry for all flexure purposes. So then evidently on the **safe** side for flexural effect, using a common set of slim-downed post-compression-based-buckled geometric properties "conservatively" for both longitudinal stress and shear stress calculations may be the easiest scheme as to avoiding the sticky database management chores if chosen so. *But structurally speaking; isn't the strategy a bit **too conservative** for flexure and **too impractical** for torsion? Also, can it be done?*

Now the **tricky part #2**:

When constructing the effective section geometry per compression-based local buckling criteria, every component needs to pass through as many evaluation sessions as needed independently with respect to the relevant axis of bending about the **principal axis**.

In an overview while modeling the cross section geometry prior to performing structural analysis, for each and every component element one should thoroughly pursuit proper answer to specific questions without getting confused:

- First and the foremost, *is this a "stiffened" element or "unstiffened" element?*
- Is this element partially effective about **X** or

- Is it partially effective about **Y**, or
- Is it fully effective about **X** or
- Is it fully effective about **Y**
- *With respect to either axis, **where exactly** is the ineffectiveness to occur?*

*Why ask? Plainly because an element in part may be ineffective for **X**-bending but is still fully effective for **Y**-bending and vice versa*

After all is set and clear to move further beyond discerning the effectiveness of the cross section, next we need to contemplate on how to organize and calculate the section properties:

Among the **FRP**, consider for unsymmetrical sections prior to computing *effective moment of inertia, the product moment of inertia, the first moment for flexural shear stress evaluation, the principal axes for the gross section and the principal axes for the effective section, etc.*, once the groundwork of relevant technical requisites was in place then ask, what is the "process flow" strategy? In addition to the readiness for elements handicapped in effectiveness against compression, how about the **shear center** location, torsional shear flow scheme and other sibling properties of **TRP**? …

Those were samples of the not-very- enjoyable steps (with traps hidden) that one needs to go through − with augmented non-engineering data management perceptions − thus obviously there were enough reasons why the suggestion of avoiding the mess caused by ineffective section (if only having choices.)

Finally and not the least, the **tricky part #3**

Barely, this is one of many questions that should be asked from those Readers to themselves if so insisted on using finite element analysis for **CRG**:

*How to make a **universal analytical model** involving both gross sections and effective sections in the same analytical session*? Think hard about this, could we make do with only one model, or must we have two or three models? How to make connection between/among many sets of output came out of multiple models?

3.16 Maximum Stresses – In Most Cases, Some Cases or All Cases

As torsion and flexure crossing their strides into each other's path dueling out damaging blows over an open sectioned member such as **CRG** from all fronts, it is crucial to substantiate "numerically" *where it hurts* the most. To us engineers handling the matter, it's already a cumbersome chore to keep tap of information streaming off the moving load analysis; then imagine the needed effort on top of which would have increased sizably if the mandates against fatigue failure were part of the design requirement.

But for better or worse as to the justification of a structure's adequacy with minimum oversights, all would come down to effective data management on the whole − especially in strategizing how the stress-related data were to be **isolated**, **grouped** and **combined** for the least

Prior to signing off any of the many intermediate steps and calling it good to go further as part of our overall design qualification process, collectively, much of the bulk of information must be sorted out for each and every stress category's extreme peak/valley values through either *absolute value, straight sum, absolute sum or vector sum* whichever scheme as applicable or appropriate no matter what class of structures were involved, **CRG** or **non-CRG**

As of this writing, **AISC Section H3.3** applies to "**Non-HSS Members Subject to Torsion and Combined Stress**." Gladly the **Commentary** to **H3.3** (most current?) has been the only Section where the catch phrase "crane girder" was officially cited (provided we haven't missed anything or that being omitted in upcoming editions) for the first time since the Green Book edition. Verbatim in part as it goes:

> "… ***covers*** *all the cases not previously covered. Examples are* ***built-up unsymmetrical crane girders*** *and many other types of odd-shaped built-up cross sections ...* ***In most cases*** *it is* ***sufficient*** *to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span.*"

There is no misunderstanding therein that the **Commentary** statements are accurate but conditionally – Readers are encouraged to extrapolate – it is not accurate for all comers but only up to "*certain extent*." And the fact is for anything beyond that "*certain extent*" and whenever tackling **CRG**s, our own judgment must step in to clear the cloud as appropriate.

Often times amid unsuspected applications, **CRG** or not, many Practitioners would let in only the face value being prompted from reading the statements such as those appeared in the **Commentary** to **H3.3** *without questioning themselves* if it works for situations whenever flexure joins hands with torsion in open sectioned members.

> Then it doesn't seem that bad sometimes or all the times, but routinely even the vastly experienced and sophisticated Engineers would fall back on those assertions as some Solid Golden Rules or Justified Excuses. Thereby as if universally safe and perfectly **OK** to be shielded behind these statements in a way granting a silent "approval" to all those passed (or failed) structural conditions short of double-checking the "structural adequacy" in much finer details, like asking frankly:
>
> > Considering longitudinal stress alone, does that (not) apply to the general state of stress combining *bending with warping*?

There is no smoke screen here or anywhere else, recalling a little while back did we (not) agree that:

> "*It pays to form a habit of being more* ***suspicious*** *or* ***paranoid*** *about anything not quite convincing*" so here it is, unless we are dealing with *simple* structures having *simple* shaped profile subject to *simple* mode of load applications (1) free from torsion's influence and (2) exempt from concern of metal fatigue, or else this could be the perfect occasion to act cautiously

Let us start with design qualification under non-fatigue initiatives such that the opening debate would cover structures of **most** ranks and applications. Anyhow, what in fact spelled out inside the core of these **AISC** Commentary statements, the "**in some cases**" expression could be a universal ambush hidden in those subtle "***in most cases***" and the "***sufficient***" remarks.

In view of an "interpreted veracity" that the "**in some cases**" expression should have applied to all structures in all applications as we normally think, correct? But, be aware of traps abound in our **CRG** realm, for which no one else but we are the ones to decide if it's necessary to exploit/defy the logics further to cover all bases in our/your design:

> The so-called "in **most** cases" certainly does not cover "**all** cases" considering all design loading scenarios − especially when effecting assessment under fatigue design mandates. And thus naturally by simple logic, *what appeared "****sufficient****" in those "****most cases****" could become "****insufficient****" in many "****other cases****" not being covered*; and that's what we have to say a few times to catch the implication
>
> No matter how we structure the linguistics at seeing those "unseen" loopholes, misunderstandings, blind spots or shortcomings, etc., *what could happen if we don't look at/into* ***all*** *the cases?*
>
> Fortunately if so, the qualification of certain class of **non**-**CRG** structure of certain geometry under certain load applications could have gotten by **OK** with the *"... In most cases it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span"* and be done with

But, one of the generic difficulties in **CRG** engineering is how not to miss anything important whether unobstructed from using design excuses or not. It is rather difficult to **quantify** the accuracy in the finger-counting of "most cases" and/or to **qualify** the definition of "**sufficient**" without skipping a beat in each and every phase as design/analysis progresses.

Thereby it is indeed difficult to arrive at a design outcome not only to satisfy ourselves but also to convince our peers or Reviewers without raising their eyebrows during their reviewing process. Anyhow, here are a few specific questions of interest and their unpretentious answers:

- Does these alleged design forgiveness hidden in the "in most cases" and "sufficient" affect all **CRGs** in this context? Yes

- Does that affect the qualification of steel structure against metal fatigue? Yes
- Does that affect structural members with unsymmetrical sections in general? Yes

Why concurring to all "Yeses" so positively would be clear very shortly. Take any generalized stick-like structure including **CRG** whether loaded through **shear center** or not, the generalized design treatment should always take in "all cases" or else as many as possible or as practical; in other words, it would lead to the all-inclusive **enveloped effects** from both flexure and torsion, of course.

On tackling the "maximum Stresses" issue for simplicity's sake, let's place focus only on flexure for now until torsion joins in for the matter later on.

Aside from handling serviceability matters and/or calculation of internal stresses, since there is no way of knowing ahead of time how the "elected/inherent cross section configuration" would work out with pleasant (satisfactory) result or not, yet knowing just to be safe, we might run through a number of mental cycles prior to committing our schematics in meeting flexural strength design requirement. Quite often during preliminary stage for that purpose, it should be more beneficial these days if the analytical result were validated qualitatively through graphics/digital-based means:

> For satisfying the "in most cases" and the "being sufficient" criteria visually, we basically eyeball for (potential) stress hot spots from the onset. With "*engineering instincts*" we rely on scanning visually "all the obvious cases" as revealed (or digitized but with limitation) in the shear diagram and the moment diagram, etc. − on that all Structural Engineers should be familiar with and be able to picture what these diagrams may look like, in general
>
> Unless the applied loads were of distributed pattern, otherwise the governing shear diagram(s) would consist of linear or piecewise straight line segments be they flat horizontal or slanted. Correspondingly by numerical integration from shear, the governing moment diagram would always comprise either piecewise slanted straight-line segments or <u>nonlinear</u> curve(s) or segments of curve(s) of higher order − at one notch over that for shear
>
> Therefore not if the loads were randomly arranged, or else the general structural response diagram(s) owing to loads of much simpler pattern could always be deduced from a number of ready-made formulas by the Books − with exception for the task of compiling <u>enveloped</u> (moment/shear) responses, usually

But with expanded intent to cover all grounds for structures designed for **moving loads** especially for those inducing more complex wheel marching patterns, typically the qualification rationales were justified on the summarized spectra built from various structural load response (or the enveloped load response diagrams derived from *influence line analyses*.) Yet it is universally true (whether for shear or moment) that there is significant difference in the "looks" of any of the individual *pre-enveloped diagrams* (in plural) and that of the *post-enveloped diagram*.

Take shear diagrams for instance:

In essence the graph from an individual point load (or a set of concentrated loads at fixed spacing) would take after the shape from connecting piecewise straight line segments involving no curves prior to being enveloped

That was before, now comes what's after:

The resultant "enveloped" shear diagram from multiple load cases after the typical "enveloping process" would no longer be linear, piecewise linear or containing straight-line segments at all, and there would be nothing but series of nonlinear curves instead

A similar algebraic-graphical feature would apply to the enveloped moment diagram as well, except that the numerical functions' order of exponent is at one notch higher. Or so we could summarize from all that:

The shapes of the enveloped response curves for either flexural shear or flexural bending moment would always be nonlinear

Before leaving the influence line subject of flexural behavior from moving loads, it is never too much or too little to evaluate how the **AISC's** "in most cases" may or may not apply. Supposing the given design wheel load nature (magnitude, distribution and/or wheel load point spacing pattern) looks quite plain (and not so suspiciously out-of-the-ordinary) then one may get by, as applicable, as follows:

In lieu of performing detailed load response envelop, the very last few formulas per **AISC Table 3-23** (as of this writing) could come in handy for flexural response due to limited number of wheel loads otherwise the **AISC** "GENERAL RULES FOR SIMPLE BEAMS CARRYING MOVING COONCENTRATED LOADS" should be helpful "in most cases" or be "sufficient" if not covering "all" cases

Understandably, indeed, it is a tedious job carrying out longhand influence-line analyses. For complex loading conditions or loads of irregular patterns, and for sure "in some cases if not most cases" through automation one could unknowingly be ensnared in a database mess once we go into enveloping phase. To get ahead in averting an out-of-control situation covering "all cases," it merits some *out-of-the-ordinary* planning on the ways and means in details such as how should the structural response data (influence line data) be enveloped when covering effects from **both flexure and torsion**.

Although minor numerical glitch in engineering calculation "in some cases or in most cases" may not affect the final design outcome that much, but it is *interesting* to point out that one of the most miscalculated quantities is from believing in a "somewhat faux but not so bad" truth that:

The *enveloped maximum flexural moment is to base on the resultant load centroid of wheel group(s) placed at the girder mid-span*, is it really?

For single-wheel load application, yes, but it's not true for loads in a group of more than one wheel. Any Readers not so convinced could confirm the unexpected fact by plotting the "enveloped" bending moment influence line for a simple case, say with only 2 wheels of unit load by running the wheel group from end to end in small incremental steps to unveil the 2 camel humps in the curve, and accordingly more humps for more wheels. What revealed should be of no major concern "in most cases" but how minor could such glitch affect the design is only relative

Not to be penny-pinching in engineering sense but to express our point herein: *the maximum moment does not always occur at mid-span*

By the same token, further "numerical" complication may be appreciated from **CRG** under torsion (as if this is not new?) Already it gives us a not very pleasant numerical-driven sensation even prior to any response-enveloping chores were started:

> Notice that all of the original torsional response equations (formulas) whether for **St. Venant** torsion moment, bi-moment or warping torsion moment had already contained multiple terms consisting mostly hyperbolic functions in terms of **β** and **βL**, thus from which *no one could humanly guess accurately even prior to the enveloping process what the final shape of any torsion-related influence line response curve may end up with*

Whether or not if drilling deep into the torsion side or the flexure side, it would take some wild imagination to envisage what the "curvature" of any "summarized/enveloped response diagram" may look like, be it for bending moment, bi-moment or flexural shear, etc. But before one can finalize the stress calculation in association with whichever enveloped (flexural or torsional) response of interest, it is always better to recognize what we are "looking at" at all stages with care;

> The "finishing shape" of the enveloped curves' curvature could be way beyond any "Normal Engineered Guesses" can come up with when comprehending the muddle either with or without good handle of **basics** in Engineering Mechanics (and Applied Engineering Math.) Under those circumstances *when all were said and done in the end, none of the shape of enveloped structural response diagrams is ever linear or ever predictable through simple algebraic contrivance*.

Given that with so many fragments or aggregates of erratic-looking "curves" thrown at us coming out of the response enveloping process (provided that the tasks were done properly/correctly,) it could pitch us off deeper into the risky ground should we not exercise the well intentioned "in most cases" warning stipulated in **AISC** with a grain of salt.

The "grain of salt" in **CRG** fatigue assessment is usually obscured in the task of figuring out the maximum stress, minimum stress, maximum tensile stress fluctuation and maximum shear stress reversal, etc. *The "worry of metal being in a state of fatigue or not" spells out the main reason why **CRG** stands out from other classes of structure because we can't do without inviting what's being **minimum** into our numerical lifecycle in addition to what's being **maximum***.

Finally without hard numbers, any Engineers who have the courage to consider the "in most cases" exclusively on only the maximum value of whether longitudinal stresses or shear stresses for **CRG** applications with an "in some cases" mentality and dare calling it "sufficient" may have to work out some serious and provable justification "in all cases."

3.17 Stress Distribution – The Hidden Intricacy

"*Guessing*" approximately (even not with pinpoint accuracy) where the extreme intensity of a specific class of stress – flexure bending stress or warping normal stress for examples − associated with a specific structural phenomenon or load response to occur at certain locality is one good skill to have or train for; thus although the state of stress can vary from node to node in general yet pretty much so to **symmetrical sectioned member** subject to simple bending, for which it is not that difficult to foretell (or guess) the stress distribution pattern on qualitative measure.

The challenge lies in the *knowing – not guessing* − by "*calculation*" of what is in it for **unsymmetrical sectioned CRG members** in the interest of more generalized state of stress.

> Pick a specific **X/Y/Z** locale, of which the intensity, orientation, uniformity in the distribution or the gradient of stress topology not only vary from one brand of stress to another but also vary from one source of load response to that of a different source
>
> With the general state of so many (at least eight) varieties of stress cropping up at a specific **X/Y/Z**, the inkling of prevalent extreme value(s) and whether or not that should occur over there as *in **most** (load) cases*, *in **some** (load) cases* or *in **all** (load) cases* is really not that clear-cut; because the applicability of these **conditional phrases** cannot be readily related in such a casual manner as hinted as in the code statement but rather **conditional** in reality

Thus the relevance of usage ascribing distinctly to only one of the **conditional phrases** involving the words such as **most, some and all** should be mutually exclusive; and whichever one was chosen to label a particular design state should always be "certified" by proper calculation

Once again, the "grain of salt" forewarned earlier in this **Chapter** fittingly applies here as well. To be on the safe side during practice, one should set a limit in applying these **conditional phrases** to specific phenomenon; and that obviously does not apply to **CRGs** but better be handled sparingly to *applicable structures* **only** to address (1) flexure-based simple-bending type of structural responses free of torsion stimulus, (2) the maximum or minimum value of certain brand of stress at a localized spot as a singular, standalone, independent and isolated entity, and (3) qualification per non-fatigue initiatives, etc.

Is the keyword "cases" appeared in **AISC** very cutting-edge or is that much unsophisticated as we understood or misunderstood? Don't answer; just think it through once more:

What about the collective **condition** over a "widespread" stress distribution/pattern of many more cases other than or beyond those as implied by **AISC's** inclusive *in most cases*, *in some cases* or *in all cases*? (By the word collective, it means not in particular regard to any individual stress category of interest but of the mixture and interaction of all stress categories combined)

Wouldn't the "on-second-thought" response to that question lead to our own suggestion next in line? But first from recalling what's in the **AISC** generic commentary:

"… *In most cases it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span."*

What should have caught our attention in this context are these **CRG application-unique** concerns and/or forewarnings as follows:

We may well entrust the **AISC**'s "*In most cases* and *sufficient*" notion as unambiguously as implicated for all normal design conditions with one exception that was never explicitly revealed, i.e. *that whole paragraph only applies to **elastic centroid-based** entities*

As simple bending implied, there is no such thing as torsion to bother with thus **independently**, and yes, the maximum shear stress and the maximum longitudinal stress do not occur at the same spot. But herein in addition to advocate that load to pass through **shear center**, it would be more helpful should **AISC** clarify further that the notion only works for structures (1) free from torsion's influence and (2) with **no** reversible loads being applied thereto. Beyond that, be careful of traps, the engineering judgement kind

A noteworthy fact of interest with respect to an open section under **flexural** influences:

The numerical value along the path taken by shear stress increases as it traverses ever closer to the elastic centroid meanwhile the longitudinal stress increases as it moves further away from the elastic centroid more so as it edges ever closer to the extremities

*With reference to the landscape of **flexure-based stress distribution** across the section profile, it is natural for locations such as (1) the **elastic centroid** and (2) all the **extremities** to be the top candidates of being the stress hot spots, thus with little doubt they became the premeditated stress hot spot locations whenever we make association with the word "maximum" or "minimum"*

Nevertheless, prior to full-heartily honoring the simple bending based notion as universal green light for **CRG** application and (blindly) relating the happenstance to all case scenarios, one should widen the focus on the implication from combined shear/longitudinal stress effects that exists elsewhere, anywhere and everywhere beyond those *premeditated locations* and see if that perception still works

First of all: *Of course it works* without flaws as some might insist on trusting the good old tribute toward these *premeditated locations* without verifying (1) does that apply to *shear center-based* stresses and (2) that the **AISC** Commentary situation only works for structures loaded through **shear center**

Secondly, that "*Of course it works*" initiative would not work as to keeping tap on the whereabouts of extreme values of longitudinal and shear stresses buried in the chaotic "numerical mess" fashioned under the unsymmetrical-sectioned **CRG** environment surrounded by all that you-name-it messes coming all together all at once from all fronts, i.e. *moving loads, bending and torsion and fatigue and effective sections, etc*. and then don't forget there is local bending stress in the top flange transpired directly from its congruous contact with the crane wheel(s)

In light of the position taken not a total exception to or disagreement with the norm, at everywhere else other than those *premeditated locations*, shall we dole out an "equal respect/attention" to what other additional kind(s) of **dependency and/or relationship** that may exist between, among or beyond the maximum and minimum shear stress(s) out of flexure and warping torsion and the maximum and minimum longitudinal stress(s) out of flexure and warping?

What happens then should one choose not to pay respect/attention? Short of calling that a sloppy science, at least some of us might be surprised if drilling the subject deeper towards its core. See for instance:

Take the effect(s) from either shear stress or longitudinal stress, with each as individual effect or combined result whether of positive or negative value, to catch on what happens at certain **X/Y** nodes/elements within a particular **Z**-section/profile or a certain zone adjacent to those so-called *premeditated* nodes, sections or profiles, etc., **sometimes** these effects or the state of combined stress may **not** dissipate or escalate from the nearby peak/valley values at gradients as rapidly or sharply as one might expect

Sounds too confusing or complicated at this juncture; and it is up to the individual to agree or not agree with the fact, but it is the fact. Unless one chooses to prove for each case through validated solid number-crunching means with no numerical prejudice and come up with a justifiable rebuttal otherwise pure rhetoric would not win the argument on the matter

Just bear in mind, our mission will not be as complete or as successful unless we get to the bottom heap of every piece of structural response data in preparation for the enveloped design spectra with reliable accuracy. And then reasonably ponder the answer to the next question:

Do we have to go through the drill with each and every **X/Y/Z** node?

Easier said than done whether doing it in full gear or not, but, the answer is yes, with a goal so as to covering all "numbers" that are exposed and those hidden or yet to be exposed throughout the multi-staged numerical process for one simple reason: *Making sure no numerical stone unturned*

To convince the point to ourselves on how could so many dynamic situations be concealed in every component element of every **Z**-cross section/profile, all we needed to do is to experience the calculation and the comparison of both longitudinal stress and shear stress at a couple of inches **X/Y** increments along the breath/depth of any selected element of interest, and then everything should become self explanatory

For some Readers only if not fully convinced by normal cerebral measure, the task involved as said as to confirming what were just mentioned in the last few paragraphs may be somewhat wearisome to carry out in practice. *It won't take much to know what to do in engineering sense, the catch is in how not to let the data management mess deter our problem-solving resolve*

3.18 Stress Distribution – The Pitfall

Even though we knew it better after all, but some may still be wondering time after time; is it universally appropriate and applicable to **all applications** by taking on the **assumption** as follows?

> Across the section profile it is **always true** that the maximum shear stress and the maximum longitudinal stress do **not** occur at the same exact **hot spot** or spots of interest when considered both as independent scalar quantity

The answer depends:

> It's only partially TRUE "in some cases," but "in most cases" FALSE for unsymmetrical sectioned **CRG** applications
>
> Why? The "in some cases TRUE state" is conditionally applicable for structures subject to only flexural influence. The "in most cases FALSE state" applies to structures subject to torsion unless the **elastic centroid** coincides with **shear center** but may still be FALSE even so. Why, again?
>
>> *Simply for the fact with respect to any open sectioned member, torsion yields its own brand of longitudinal stress, and not just one but two varieties of shear stress; so we need to know what kind of shear is in our focus – remember torsion comes with three different varieties of stress*

So then, is **AISC** commentary incorrect in this regard?

> No, not really; but our answers could also be Yes and No depending on how thoughtfully that was interpreted; **AISC**'s intent is entirely accurate only that (1) the structure is loaded through **Shear Center** by the "simple bending rule" and (2) by which we, the Engineers, do follow strictly the simple bending rule's limitation in qualifying our structures accordingly – or else we are wrong

To make it much clearer, by observing the "simple bending rule" per **AISC** we automatically cast its constraint on dealing with **elastic centroid-based** stresses only. If not then we tread into the territory jam-packed with **shear center-stimulated** inconveniences.

> And in that case we are definitely on our own as of this writing;
>
> Then can we pinpoint where is (are) the torsion-stimulated hot spot(s) outside of the usual markings characterized under flexure? Or be more straightforward to address what is most critical in terms of the correctness of our engineered results, of which we should ask especially for unsymmetrical sections:
>
>> Where exactly is the **elastic centroid**?
>> Where exactly is the **shear center**?
>
> Even though ahead of time we might possess a perfect data management plan and a perfect numerical processing scheme; but if the definition of coordinates of either one of these nuclei were off (garbage in) then everything we do will be off (garbage out)

Following an "initial pass" of calculation, certainly it's not all but a good stopping point to *review* (study) the results prior to defending the structural adequacy against all odds. For **CRGs** it involves both fatigue and non-fatigue drills – each being qualified in separate sessions − one must keep a longwinded account on all probable numerical ambiguities by raising a few questions on our own first, for examples:

(a) What is the state of longitudinal stress and/or shear stress at a certain intermittent stitch weld, which is only an inch (two, three or a few more inches) away from some of the premeditated or those more popular "hot spots"?

(b) How may the shear stress and longitudinal stress (i.e. non-fatigue stress and fatigue stress reversals) interact with each other if not only over these hot spots of interest but also elsewhere nearby?

(c) For either longitudinal or shear stress, has the variation, overlapping and attenuation in the stress distribution patterns between flexural and torsion been assessed?

There should be more questions during the *review* than these samples if digging deep enough not for argument's sake. But regardless, all that was "interesting" enough to discourage using any of the water-downed schemes or fakery through valorous guessing or unconditioned engineering innocence during **CRG** design debugging/reviewing process. Other than that but:

> Shouldn't we agree that the **actual** dissemination of stress, whether as standalone or as combined entity, is indeed much more complex than it appeared? The answer is yes, but then what?
>
> Take the **X/Y** spread across **any Z**-positioned profile, whether at profile's interior nodes or at unstiffened components' extremities, regardless to how the distribution of shear stresses and/or longitudinal stresses that may vary, attenuate, intensify or interact in whichever pattern, but in the end our calculation should "always" prove that the cross section (at all **X/Y** spreads) is *free from being overstressed at all locations* (hot spots or not) whereas as to meeting normal Quality Assurance Intent ahead of reaching consensus on the adequacy of **CRG** duly from evaluation of internal stresses' viewpoint
>
> Put that in other words, everyone in the Engineering Team (both the Designer and the Reviewer) had to sign off attesting that there is no overstressed condition subsisted anywhere and everywhere

Knowing that we are on our own once driven away from simple bending, but in concept, things could still be streamlined without blessing from anyone but us if we see it through with nothing fancy but the basics. There is no big puzzle there but kind of trivial in the stress distribution matter that may be of interest; one recognizes that, at any series of "nodes of interest" off any component element, provided stresses were measured along the plane passing through the element's **mid-thickness** traversing the element's breath/length:

- *The variation in distribution for both flexural bending stress and warping normal stress in their raw form does follow **linear** pattern*

- *However, the distribution of shear stresses always follows **nonlinear** pattern (excluding **St. Venant** pure shear)*

Finally, if treating the sum of aggregate stress as algebraic addition then:

- The consolidated longitudinal stress, expressed as $\mathbf{f_L}$, could be generalized as the sum of axial stress $\mathbf{f_a}$, bi-axial bending stress $\mathbf{f_{bx}}$ and $\mathbf{f_{by}}$ and the warping normal stress $\boldsymbol{\sigma_n}$

- The consolidated shear stress, $\mathbf{f_V}$, could be generalized as the sum of flexural shear $\boldsymbol{\upsilon_x}$ and $\boldsymbol{\upsilon_y}$, pure shear stress $\boldsymbol{\tau_0}$ plus the warping shear stress $\boldsymbol{\tau}$

Since we are not dealing with conventional structures subject to conventional load – not under **AISC** simple bending rule − thus if being asked, one may need to contemplate briefly before giving answer to this important question:

*With that many participating stress vectors − f_a, f_{bx}, f_{by}, σ_n, υ_x, υ_y, τ_0 and τ converging upon each and every **X/Y** node dispersing across a **CRG** section, among which, some are positive and some negative, some at bolt holes and some near weld elements or at plain base metal − should it be **OK** or not to "separately evaluate" or "isolate" the longitudinal stress category from the shear stress category and then qualify each category by its respective allowable stress?*

<u>If the self-approved answer is "Yes"</u> then

AISC sections **H3.3**(a) and **H3.3**(b) may apply, or does it really? It is up to the Engineers (of record) to justify the relevance of the "in most cases" remark appeared in the Commentary; and certainly the "on-our/your-own rule" would perfectly apply to the situation as well

Meanwhile to whatever that matters to structural safety, it may not be **OK** once the peer Reviewer and any independent Reviewer disagrees with our/your work. <u>However, just keep in mind such agreement (or non-agreement) does not guarantee a genuine **OK** in reality</u>

<u>On the other hand if the answer is "No"</u> then

We are not quite finished with that "simple" question yet. There would be further issues regarding the treatment to consolidated longitudinal stress $\mathbf{f_L}$ and the consolidated shear stress $\mathbf{f_V}$:

- How should we resolve their interaction or their combined effects in general?
- How should we handle the effect combining both $\mathbf{f_L}$ and $\mathbf{f_V}$ in design of **CRG**?

3.19 Combined Stresses

As we qualify <u>conventional **non-CRG** structural members</u> meeting <u>**non-fatigue**</u> strength requirement, it might have been "*sufficient in most cases*" to address "only" the required stress/strength established for the specific load natures and the associated load combinations, and that's it. That is, "*in <u>most</u> cases*" we meant.

Under "**simple bending rule**" the "underwritten value" in the respective set of allowable stresses − whether for longitudinal tensile stress or for shear stress − each would remain constant throughout the entire cross section while qualifying on behalf of non-fatigue-critical conditions (*unless hybrid materials were used in the same cross section*.)

In other words, longitudinal stress – tension only, Readers to tell why so − is qualified per allowable longitudinal stress of one unique quantity, likewise shear stress is qualified per allowable shear stress of a unique but different quantity; it works *usually* as we discount the fact/assumption that whether the two types of stress overlap/interact or not

*But what happens then within a certain cross section's confine, if longitudinal stress and shear stress do overlap and interact as suspected as if they join up to dwindle the invulnerability of the structure such as **CRGs** against all odds?*

Where there's need of considering interaction between simple stress vectors, some may choose to do nothing fancy while others may go with full-blown Theory of Elasticity way or the extreme Engineering Mechanics **Von Mises** way. Easier said in the full-blown instance, but only if there is no concern of metal fatigue. Why?

Because there is no (easy) practical way to consolidate the tensile stress fluctuation and shear stress reversal extracted from moving loads application − in one take − to facilitate fatigue assessment by those tactics, and even in the non-fatigue drill, isn't it beyond being "practical" for **CRG** applications, or would that invite further challenges in the provision of **shear center-based principal stresses** with respect to principal planes in **3-D** before arguing over how to qualify the structure against fatigue failure?

For fatigue strength assessment, merely identifying where a certain brand of maximum stress might take place at a certain global **Z** location is not enough.

> What also needed are the multitudes of maximum stress and minimum stress occurrence through exhaustive computation for each and every **X/Y/Z** node of interest wherever as required for stress fluctuation/reversal calculation. Knowing the added difficulty stemming from understanding of what's involved with structural design against fatigue for a fact, Readers are encouraged to find out if **AISC Commentary** to **H3.3** still makes sense (or not) in this context on behalf of **Crane Runway Girders**

Let's face it, not knowing beforehand by which effect (flexure, torsion or metal fatigue) or combination of multiple effects could dominate our design of so many key components, it would be impossible for anyone to "guess" the precise whereabouts of a local **X/Y/Z** node where its absolute maximum and the absolute minimum stress might control if taking into consideration the *overlapping, canceling, enhancing and fluctuating* between longitudinal stress and shear stress.

Some of the very seasoned experts or laymen alike, familiar mostly with how **non-CRG** structures were qualified, tend to practice through their purported professional hindsight to sign off the design adequacy of **CRG** structures (through 100% **non-CRG** frame of mind) yet with no clue of globally the whereabouts of stress hot spots and what their true state of extreme (numerical) circumstances are.

> The bottom line of all that was akin to getting it over with without going through all the necessary qualification steps which were critical to **CRG's** lasting endurance. Actually it is so much more than by following **AISC** simple bending rules, or by blindly trusting that the maximum bending moment occurs at the mid-span and maximum shear occurs at the support, etc. Yet the best (worst) part for taking care of **CRG** is anything but − **The beauty of CRG is, every X/Y/Z could become a stress hot spot in its own right, if we've missed it then we missed it**
>
> Important thing is, the so-called hot spot could be any node of interest hidden or exposed at anywhere. Without a detailed analysis, there is no easy way to tell would the hot spot be perfectly **OK** or failed by (1) material yielding or (2) fatigue due to tensile stress fluctuation or (3) fatigue due to shear stress reversal, or (4) *all three conditions*

Reasonably speaking, it would be up to the Engineers to either "determine" or to "experience" where "exactly" these stress hot spots or cold spots really are, and that is never easy.

The message: Don't do it by second guessing. But one thing for sure: the longitudinal stress and the shear stress are always acting in perpendicular to each other at all **X/Y/Z** spots be they located wherever within the thin-plated open profile. Therefore provided that the subject of interest is not for fatigue assessment's sake then it should be reasonable for practical purpose to simplify our chores as follows:

(a) Among however many methods of combination, it may be more intuitive for orthogonal stress components to form the simple **SRSS** stresses = **SQRT** ($\mathbf{f_L}^2 + \mathbf{f_V}^2$) as the resultant stress and then

(b) Compare it with the material yield stress **Fy** (with modification by either $\mathbf{\Phi_T}$ or $\mathbf{\Omega_T}$ whichever is appropriate per **AISC H3.3**)

The very last paragraph has just summarized the recommended treatment to non-fatigue stresses. With that for any cross section profiles being local-buckling free, the efforts taken to meeting non-fatigue design requirement now becomes much more straightforward.

3.20 Fatigue Stress Ranges

How privileged that would be if only metal fatigue is not a share of our design concern.

Yet not as wished for by that at all in reality, metal fatigue is a genuine concern − considering the goal for **CRGs** in the Mills to live on for as long as anticipated in pursuance of uninterrupted operation. To prevent metal fatigue the best we could do as obliged, simply we must go along with as many dreary steps as needed in order to meet the design intent per limit state of fatigue – basically not only to qualify the adequacy of global structural member but also its many local components, too

When structures were qualified on "stress criteria" that deals with scalar entities carrying with dimensional units as in ***psi*** or ***ksi***, etc., it is important to understand what "Fatigue Stress Range" really is before fulfilling our design obligations in this regard. Some of the "basic" distinctions/definitions of critical design state(s) applicable to both fatigue and non-fatigue purposes could be summarized as follows:

- Non-fatigue condition:

  For all structures whether living a fatigue-challenged way of life or not, needless to say the as-calculated extreme numerical peak **or** valley (longitudinal or shear) varies from node to node. As simple as that but here are the blurry parts not so confusing but should broaden the norm to certain extent;

  Take the subject of nodal (local) *longitudinal stress* for instance,

  > Albeit a "peak" being the maximum numerical value associated more often with tensile stress (being positive) but that could also be compressive stress (being negative) if the node in focus never sees tension. Likewise a "valley" usually represents the minimum value more so with compressive stress (being negative) but that could be tensile stress (being positive) if the node is always under tension.
  >
  > Globally, a system-wide extreme value (peak or valley) implies that there is only one such value among all nodes/elements subsist in the corresponding stress domain (tension and/or compression.) It doesn't matter where is the **X/Y/Z** for which that value may occur at, and regardless if that highest value had occurred only once, twice or however many times it registered during the structure's lifespan. Whenever compression presents, what more to worry is local buckling

- Fatigue condition:

  One of the essential tasks on qualification against metal fatigue comes under the *strategically* "dealing" with the historical account on the ±range of stress *intermingled* among all participating numerical entities in the domain; the nature of "dealings" is entirely different from that on non-fatigue counterpart

  For each node, all numbers must be "tracked, handled and ranked" – explained in upcoming **Chapters** − one by one no matter at long last some may become peaks, valleys or somewhere in between. That is, every piece of raw information (of fatigue significance) was evaluated painstakingly at all profile slices from **z = 0** at one end to **z = L** at other end for all probable as-modelled **X/Y/Z** nodes prior to "finalizing" node-wise fatigue stress assessment

  To a specific **X/Y/Z** nodal location, the critical value for fatigue assessment is the all-time highest stress range as coalesced at the matching **X/Y/Z** coordinate, *which, however, is meaningful only if the number of* ***cycles of live load-induced shear stress reversal and/or tensile stress fluctuation*** *throughout the structure's existence were also given*

For design against fatigue, it is not viable to be granted a universal **constant** fatigue allowable stress range across the board for all nodes to share at each and every cross section profile throughout the entire structure. The hint: instead of one single constant, we have to (or must) "secure" as many connection-sensitive allowable stress values as required in design. To further into the subject, one should understand a few facts:

- Perfection: Though rarely or *never exist in reality*, yet only theoretically true unless the structural member is a non-composite/non-built-up rolled section milled perfectly free from any intentional or incidental defects during the milling/fabrication process, then there would be **only one** allowable stress value for all – but that's not realistic

- Imperfection: Conversely defects abound in real world. The so-called defects in this context were not innate or natural from the (steel) material itself but were unavoidable as result by-design/detailing and by-fabrication or inherent from other artificial means − most of which were attributed to drilled holes or weldments at interfacing connections, supports, mechanical attachments or stiffeners or the likes found mostly at locations on tactical purposes within any cross section along the girder span from end to end

Other than from unforeseeable natural sources, structures could fail unexpected or doomed to fail as result of careless design. A few cases in point: (1) being too naive to fulfil the full-fledged engineering duty either did nothing, not enough or qualified the design based on haphazard judgement or (2) being too ignorant of the consequence from not dealing with fatigue in proper manners should be the biggest contributors to most **CRG** failures.

> To be more specific, blunders from blessing the structural adequacy solely on the maximum longitudinal/shear stress of base metal at extreme fibers/profile interior nodes without due consideration to the base metal at nearby bolt holes and/or weldments, etc. had been one of the most common design glitches

It is always a design catch from not realizing that the allowable stress range values could be drastically different among adjacent localities barely a couple of inches or even a fraction of an inch apart. But unfortunately so, its ramification to design process is one of the most overlooked matters in structural qualification per fatigue mandate.

One should have "recognized" by now that fatigue design assessment is meaningful only if the number of live load application cycles were also given; thereby our next suggestion or word of warning: Don't miscount the number of fatigue live load cycles involving stress fluctuation/reversal. Otherwise any form of "miscounting" would definitely lead to a bogus garbage-in-garbage-out situation. For **CRG**, *failure due to the garbage in-out effect at any "structural component" of practical interest (or of no interest) is equivalent to failing the entire design application.* But how dare anyone say that?

Correct count or miscount of cycles is not a very clear-cut conversation piece, sometimes or most of times; there could be several varieties of that to watch out for. Depending on the objectives, different Design Guides, Design Criteria, Standards, Codes or Committees, whether having direct or indirect influence on the design of "our" **CRG** or not, may have different implication on the so-called "cycles."

Some of the Committees associate their "duty cycles" closely with operation-related timing frequency, rated capacity, lifting heights, etc. These said parameters were more important with respect to Mechanical and Material Handling significances. Such so-called "cycles" might not be so meaningful "structurally" for *component-based* fatigue strength evaluation. The equivocally sound-alike/look-alike term of "cycle" could even be misleading to **Crane Runway Girder** design applications if abiding by faithfully without prior knowledge as a whole of what kind of base metal structure is, or how the stress flows, and which local component or attachment or what type of connection details look like, etc. were in for our assessment.

For steel design in particular of **CRG**, the definition of "fatigue load cycle" making sense to Structural Engineering purposes could best be interpreted per latest **AISC Appendix 3** and its **Commentary**.

The true Structural Engineering-based "Fatigue Load Cycle" hardly has anything to do with the Mechanical Engineering-based "duty cycle" but it has a lot to do with specific (connection) details in correlation with the "*number of service live load applications inducing stresses within the elastic range*" of:

- Repeated **stress fluctuation** involving **tension**, or
- Repeated **stress reversal** in **shear**

Or on the flip side at any specific component, connection or interface as detailed, for which provided by both conditions as follow were "proven" true by calculation:

- If there is no **stress fluctuation** involving **tension** and
- If there is no **stress reversal** in **shear**

So then the applicable "structural live load cycle" for that specific component, connection or interface would = 0 no matter how many "mechanical duty cycle" there imparted. Readers are inspired to look deeper how can cycle = 0 be possible for any **CRG** to relish in any Mills then ask this question:

What could be the best design outcome of all, both structurally and mechanically?

By latest **AISC** specification as of this writing, if the calculated tensile **stress fluctuation**/shear **stress reversal** (**SFR**) in every component of the structure were always below the applicable **AISC** fatigue threshold stress $\mathbf{F_{th}}$ without causing fracture then an unlimited number of load cycles may apply − as the best outcome.

On the other hand, at any given **X/Y/Z** nodal component whenever the **SFR** > the applicable $\mathbf{F_{th}}$, it would be predestined as **NG** or as failed in the old days for *being overstressed* when the conditions were assessed by our olden design mentality following the classic older Code standards. But then, it is not the end of the world if evaluating the same situations per current **AISC** specification as long as the estimated number of live load on/off cycles allowed per **SFR** were not exceeded − funny but not so funny when using the key phrase, estimated number.

Certainly the current **AISC** specification does not grant an outright certificate declaring an unconditional **OK** or **NG** whenever **SFR** > $\mathbf{F_{th}}$ but instead it gives Engineers a fair chance of going backward to reevaluate how many live load on/off cycles are allowed before an imminent mishap (initiating crack nearby a certain component where **SFR** > $\mathbf{F_{th}}$.)

> Under the **SFR** > $\mathbf{F_{th}}$ circumstances by following the newer decree, at least it calms down the fear of the structure being handed a guilty verdict under the "old rules" whereas in the mean time the structure is given an opportunity by the "new guidelines" on how to survive if sticking by the rule of not to exceed the remaining number of fatigue "cycles" so long as the base metal crack(s) has not been initiated yet

Finally could we safely say that the key issue in the design for limit state of fatigue beyond the threshold stress $\mathbf{F_{th}}$ now rests in the accuracy in the estimated number of design live load on/off cycles; isn't it?

3.21 Live Load On/Off Cycles – The Exact Count

Whether the process of structural steel design qualification these days has become much more complicated or simplified is rather debatable depending on what design criteria were imposed and what design guidelines were followed; but regardless, at least on fatigue failure- related matter, how to estimate a universal fatigue load cycle count for the engineering of specific **CRG** application could be very interesting or rather tricky.

Again, just beware of a garbage-in-garbage-out trap buried somewhere – or everywhere as matter of fact in this regard – so be careful. Before being at variance with anyone else on the “cycle count issue” we would be better off to be self-challenged by considering the implication from as many scenarios as we could. If we had not gone hardy nutty on the matter of concern just yet then here are a few interesting – yet rather drawn-out – questions that needed answer for:

- Assuming an overhead crane came into the active girder bay doing **one** of its **mechanical** “duty cycles,” by which the crane bridge was parked at a stationary position the whole time or was kept in place within the girder span with little (or no) movement along the **Z**-axis. Meanwhile, if the trolley kept moving to and fro, did its usual lifting, dragging, flinging, loading and unloading for countless “cycles” along **X-** and/or **Y-** axes, so then:

  How many cycles should be counted for **structural** design consideration out of this “one” **mechanical** duty cycle?

  Should be counted as one, since the bridge visited the **CRG** span only once, or else be counted as many cycles as the number of rackets caused by the trolley hurling actions?

  The *answer could be “yes” or “no” to all or nothing since regardless to however many times trolley moved, we must evaluate the number of cycles as “variable” based on the counts from tensile **stress fluctuation/**shear **stress reversal** actually induced at **each** “local **X/Y/Z** location” and not merely by the global number of commotions realized, see a perfect excuse?*

- In steps, start from prescribing generic loading/unloading traits then complementing from that into sequential wheel load marching patterns, soon as load conditions were set and analysis was in progress or completed;

  We might run into special situations in that some of the structural components “**always**” experience shear reversal but occasionally with and/or without a (concurrent) tensile stress fluctuation, meanwhile some other components “**never**” experience either/neither all or any of wavering in stress assimilating change in numerical sign. To sum up the extreme conditions, it’s either always or never

  What becomes trickier is that at certain structural component, there is always a probability of some acute loading action/setting, which might lead into an extreme maximum stress followed by unloading action/setting reverting into an extreme minimum stress situation.

  Then in view of the result as such, seemingly there is an **in-phase/out-of-phase** loading problem or load response problem. So how many variations in cycles should be assigned for each discrete component within the same cross section of interest? How about the many cross sections ranging from **z** = 0 to **z** = **L**?

  Should it be constant number of cycles uniformly applied to the entire girder or **variable** number of cycles applicable to different parts of the girder, and how to pin down the **variable**? How to segregate the “**always**” and the “**never**” occurrences then determine the tensile fluctuation influence and the shear reversal influence? …

- *If a critical component experiences shear stress reversal at one instant and tensile stress fluctuation a moment later or back and forth, the happenstance is much like exhibiting the two sides of the same coin*

  Since the “Code” does not deal directly into subject of “multi-axial” fatigue condition based on combined bending and torsion, should the loading cycle be counted twice or once or more? See the loophole or a problem instead?

- Although it may be an infrequent setup, but for some framing systems that consist of (1) continuous **CRG** in length covering more than one span or having (2) simple-supported simple-spanned members yet propped with knee brace at either/both ends leading into multiple continuous spans or segments, in "normal" practice situations like these, the setup may appear logical with respect to reducing the deformation or cutting down non-fatigue-critical stress magnitudes; however, how does anyone account for the fatigue-inducing effect from load cycles originated from the girder in the *adjacent span and beyond* (due to bending moment distribution and/or bi-moment distribution, has anyone thought about what to do with situations like that?)

For certain some of the Readers are keen on counting the cycles more carefully – but may very well learn that it is much harder than it seemed. Finally if there is not enough inspiration in our **CRG** from hereinbefore then the upcoming **Chapters** could be much more interesting.