IACS Unified Requirements for Polar Ships

Review of Tripping Requirements

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1. Introduction

The IACS Unified Requirement for Polar Ships (PUR) is in its final stage of development. The draft UR contains local buckling requirements pertaining to web buckling/crippling and flange buckling. At the moment there are no explicit tripping requirements. The aim of this report is to address the tripping requirements.

The PUR employ plastic limit states in the definition of ice strength requirements. The frame design load is a patch load acting normal to the shell plating. The intended plastic behavior of frames designed to the PUR is two-fold. We have set the design point to be well above initial yield, but yet at a point where the permanent plastic distortions remain quite small. For loads above the design point the deformations grow quite quickly. Nevertheless, even at large deformations, there is slowly increasing load-bearing capacity, meaning that there is a large reserve of capacity above the design point.

The PUR design load will tend to cause bending of the frame, with the bending resulting in tension in most of the upper portion of web and flange. In such cases overall tripping would be unlikely or even impossible (analogy – a column in tension can not buckle). However, at large deformations, tripping has been observed, even in the case of symmetrical sections. Figure 1 illustrates the locations of tension and compression for both small and large deflections. As the deformation grows large, the region of tension grows to include almost all the section. Figure 2. Illustrates the type of tripping that can occur at large deflections, even as the tripped region is in tension.

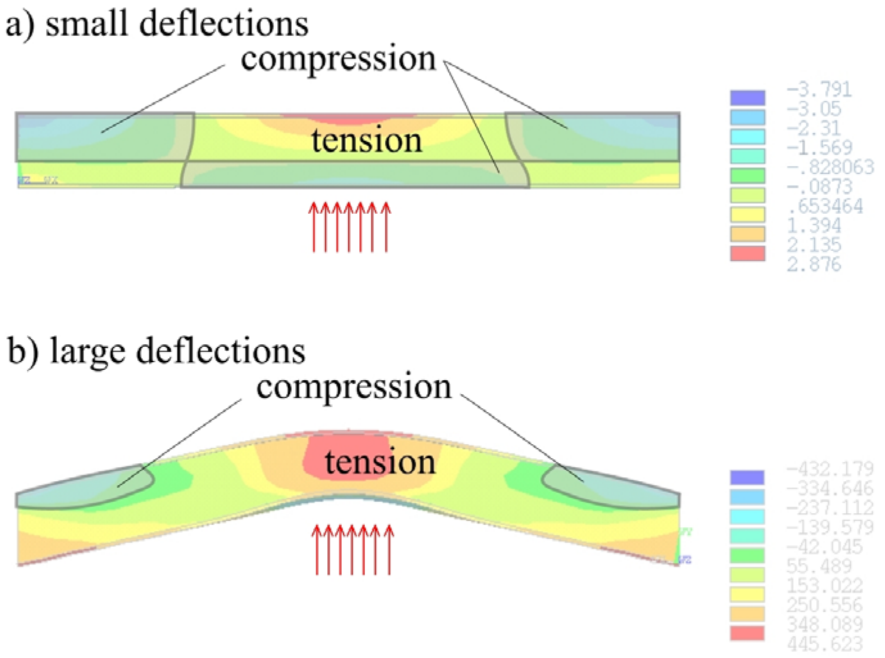

Figure 1. Growth of tension zones in laterally loaded frame.

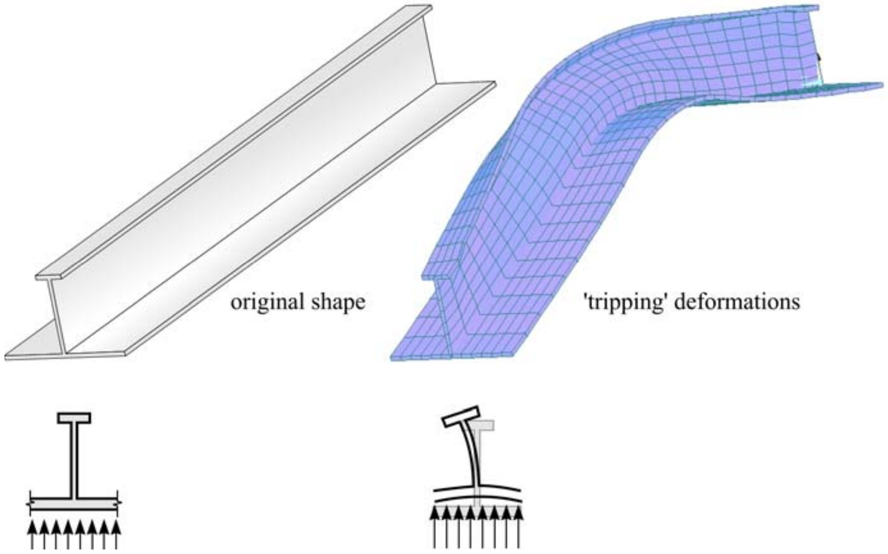

Figure 2. Tripping of laterally loaded frames at large deflections

A related issue is the behavior of unsymmetrical sections, mainly angle sections as well as bulbs and tilted sections. Unsymmetrical sections will tend to distort laterally (trip) even at small deflections, due to the unbalanced lateral moment that occurs due to the asymmetry of the shear center of the section. With regard to this, there are two key issues addressed in this report. These are:

1) do unsymmetrical sections show a significant loss of capacity at the design load (relative to symmetrical sections of the same modulus), as a result of a ‘tripping’ behavior.
2) do any sections show a significant loss of reserve capacity, relative to other sections

2. Tripping Analysis

The frames listed in Tables 1,2,3 were analyzed with ANSYS to examine the range of behavior up to the nominal design point and beyond. The Annex shows a typical ANSYS input file showing geometry, loads and non-linear solution control. Three types of frames were analyzed – tee, angle and flat bar sections. Figure 3 shows the sections of the angle frames. Figure 4 shows the load vs. deflection plots for the four angle sections. Figure 5 shows the sections of the tee frames. Figure 7 shows the load vs. deflection plots for the four tee sections. Figure 8 shows the sections of the flat bar frames. Figure 9 shows the load vs. deflection plots for the four flat bar sections. Figures 4,7 and 9 also contain the calculated PUR capacity values (the rule values).

Table 1. Properties of Angle Frames Analyzed

Item [units]	**value(s)**	**value(s)**	**value(s)**	**value(s)**
file	a1_0d	a1_5d	a1_10d	a1_20d
frame type	angle	angle	angle	angle
frame length [mm]	L=2000	L=2000	L=2000	L=2000
load length [mm]	b=500	b=500	b=500	b=500
web height [mm]	hw=200	hw=200	hw=200	hw=200
flange width [mm]	bf=75	bf=75	bf=75	bf=75
frame spacing [mm]	s=350	s=350	s=350	s=350
web thickness [mm]	tw=8	tw=8	tw=8	tw=8
plate thickness [mm]	tp=10	tp=10	tp=10	tp=10
flange thickness [mm]	tf=10	tf=10	tf=10	tf=10
Elastic Modulus [MPa]	E=207000	E=207000	E=207000	E=207000
yield strength [MPa]	fy=300	fy=300	fy=300	fy=300
Tangent Modulus [MPa]	Et=500	Et=500	Et=500	Et=500
inclination [deg.]	phi=0	phi=5	phi=10	phi=20

Table 2. Properties of Tee Frames Analyzed

Item [units]	**value(s)**	**value(s)**	**value(s)**	**value(s)**
file	t1_0d	t1_5d	t1_10d	t1_20d
frame type	angle	angle	angle	angle
frame length [mm]	L=2000	L=2000	L=2000	L=2000
load length [mm]	b=500	b=500	b=500	b=500
web height [mm]	hw=200	hw=200	hw=200	hw=200
flange width [mm]	bf=75	bf=75	bf=75	bf=75
frame spacing [mm]	s=350	s=350	s=350	s=350
web thickness [mm]	tw=8	tw=8	tw=8	tw=8
plate thickness [mm]	tp=10	tp=10	tp=10	tp=10
flange thickness [mm]	tf=10	tf=10	tf=10	tf=10
Elastic Modulus [MPa]	E=207000	E=207000	E=207000	E=207000
yield strength [MPa]	fy=300	fy=300	fy=300	fy=300
Tangent Modulus [MPa]	Et=500	Et=500	Et=500	Et=500
inclination [deg.]	**phi=0**	**phi=5**	**phi=10**	**phi=20**

Table 3. Properties of Flat Bar Frames Analyzed

Item [units]	**value(s)**	**value(s)**	**value(s)**	**value(s)**
file	f1_0d	f1_5d	f1_10d	f1_20d
frame type	angle	angle	angle	angle
frame length [mm]	L=2000	L=2000	L=2000	L=2000
load length [mm]	b=500	b=500	b=500	b=500
web height [mm]	hw=200	hw=200	hw=200	hw=200
frame spacing [mm]	s=350	s=350	s=350	s=350
web thickness [mm]	tw=8	tw=8	tw=8	tw=8
plate thickness [mm]	tp=10	tp=10	tp=10	tp=10
Elastic Modulus [MPa]	E=207000	E=207000	E=207000	E=207000
yield strength [MPa]	fy=300	fy=300	fy=300	fy=300
Tangent Modulus [MPa]	Et=500	Et=500	Et=500	Et=500
inclination [deg.]	phi=0	phi=5	phi=10	phi=20

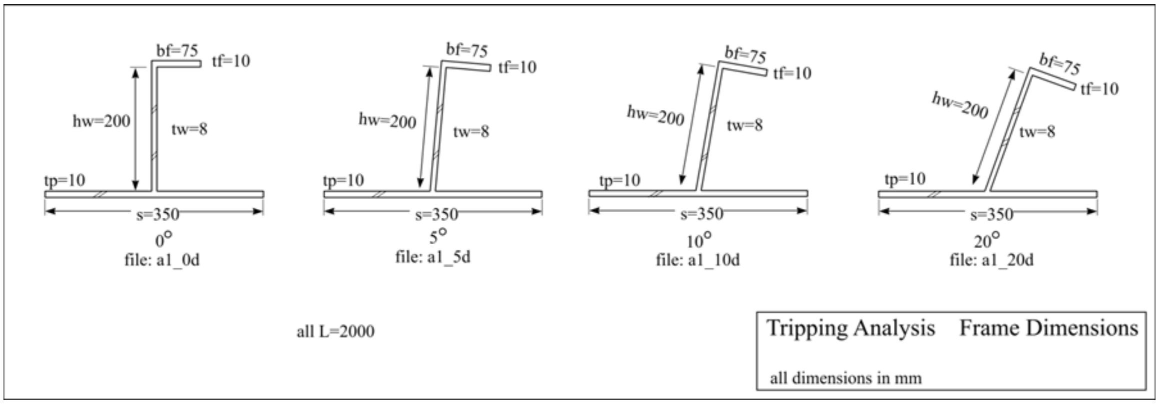

Figure 3. Angle sections analyzed.

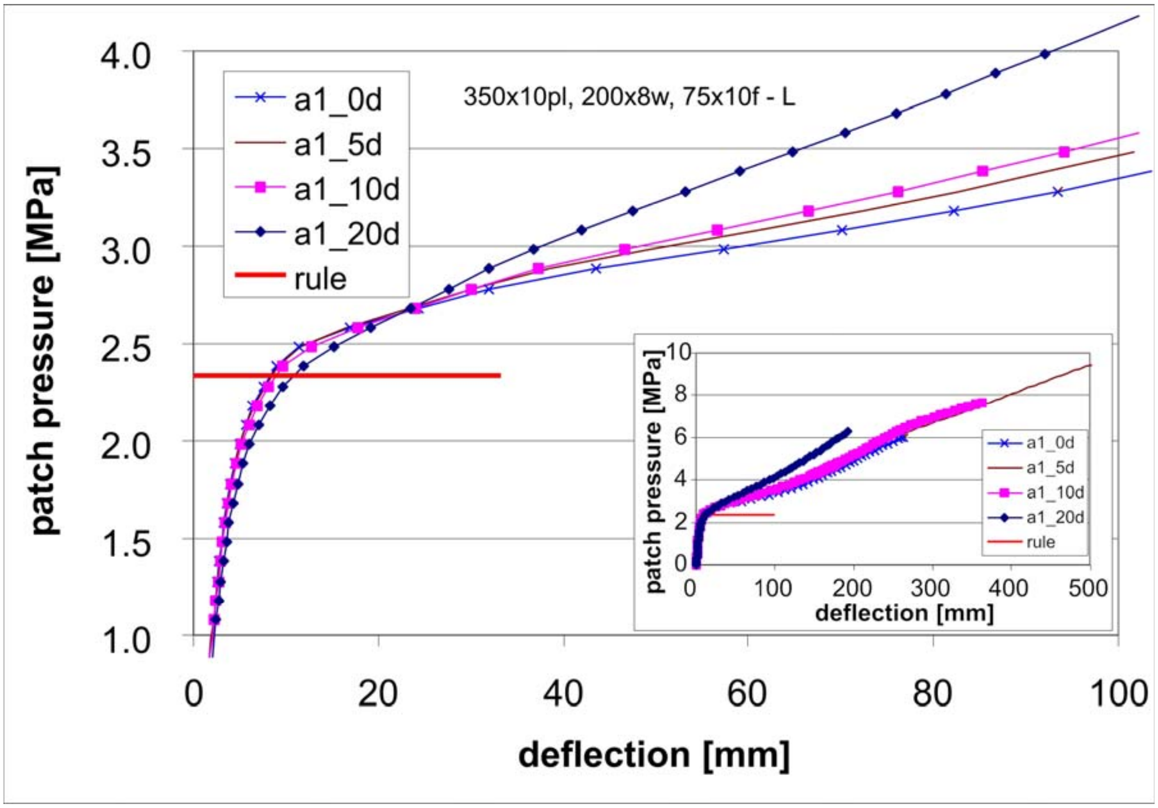

Figure 4. Load vs. deflection for angle sections.

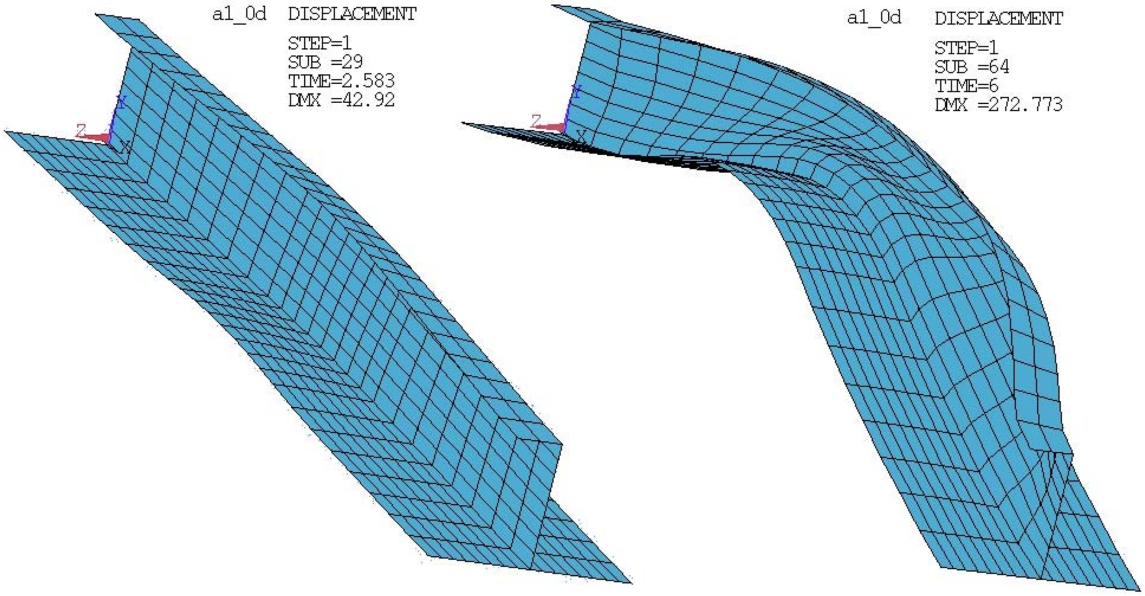

Figure 5. Angle section deflections at 2.5 MPa and 6 MPa.

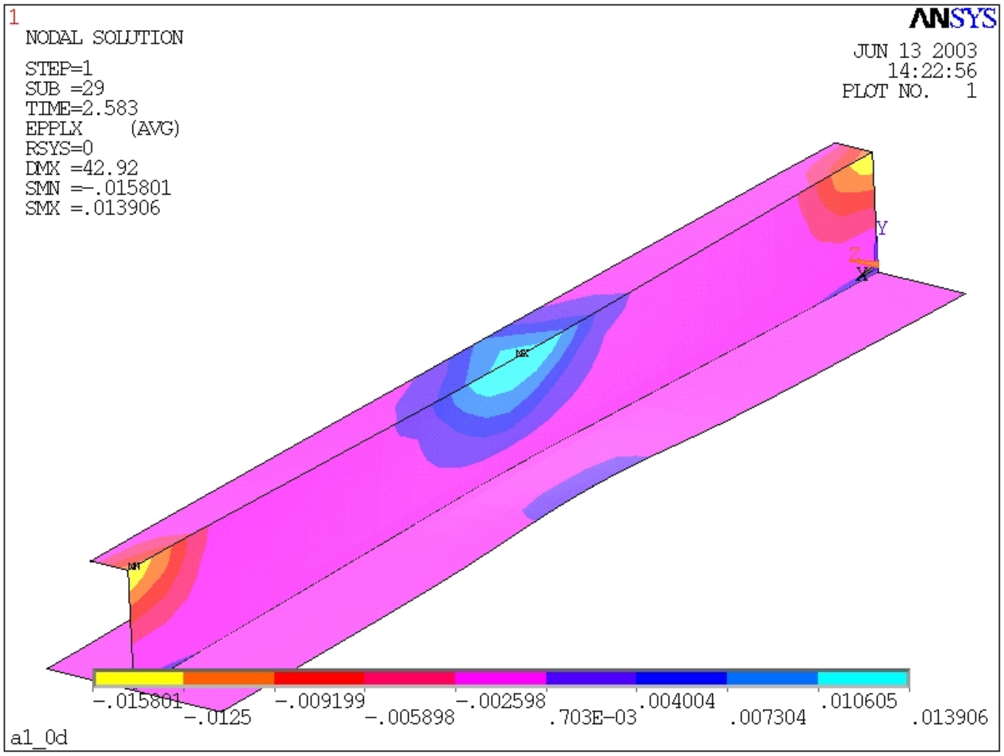

Figure 6. Angle section strains at 2.5 MPa.

Figure 6 illustrates the asymmetrical stresses in the flange of an angle frame. This cases shows the x-direction plastic strain in the member, at a load just above the design load.

Note that that while the stresses in the flange vary from the web to the free edge, there is no reversal of the strain. The asymmetry exists, but does not significantly impair the elastic or plastic behavior of the frame. It appears that the beneficial affect of the tensile stresses in the deformed section, overcomes the negative effects of the asymmetry.

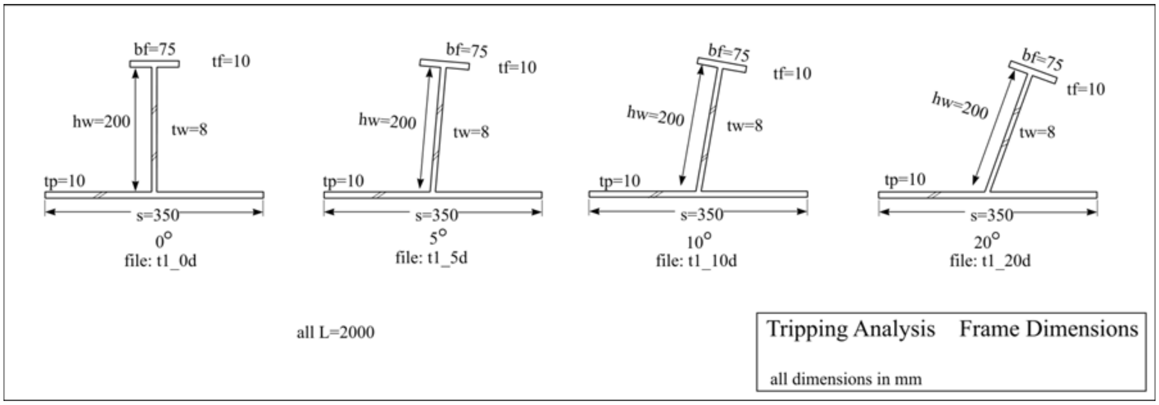

Figure 7. Tee sections analyzed.

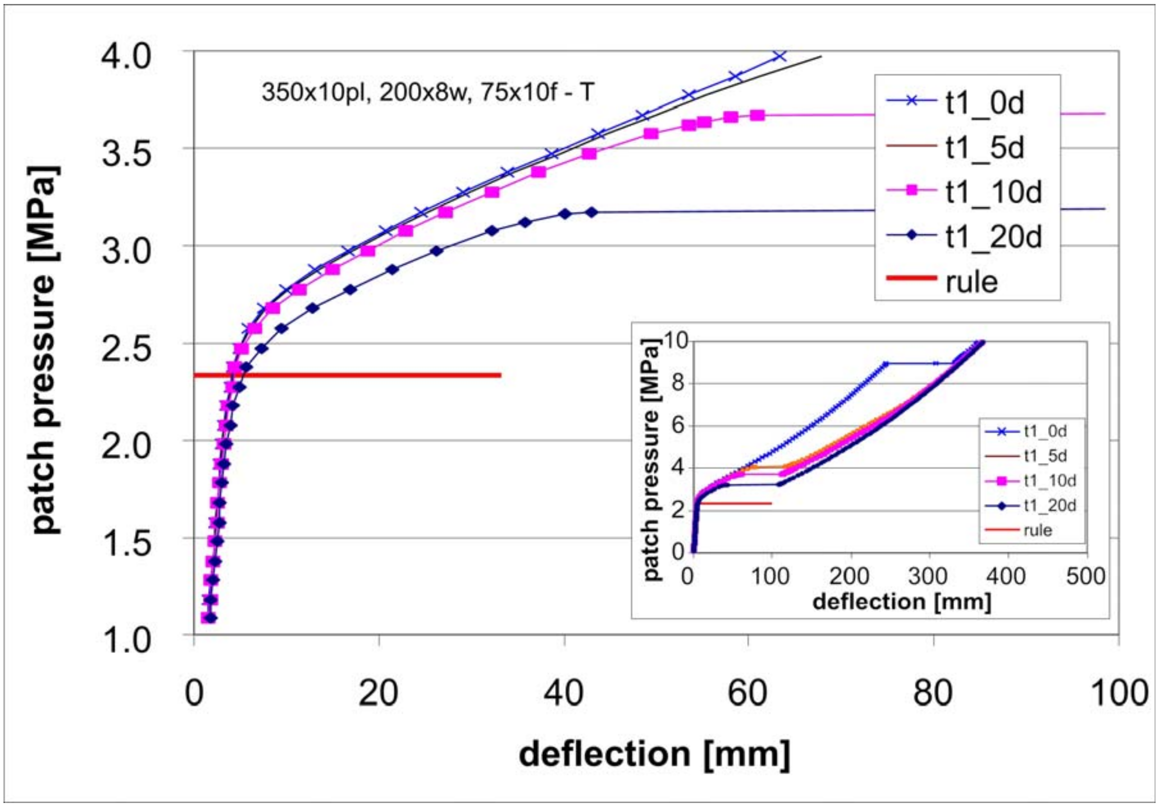

Figure 8. Load vs. deflection for tee sections.

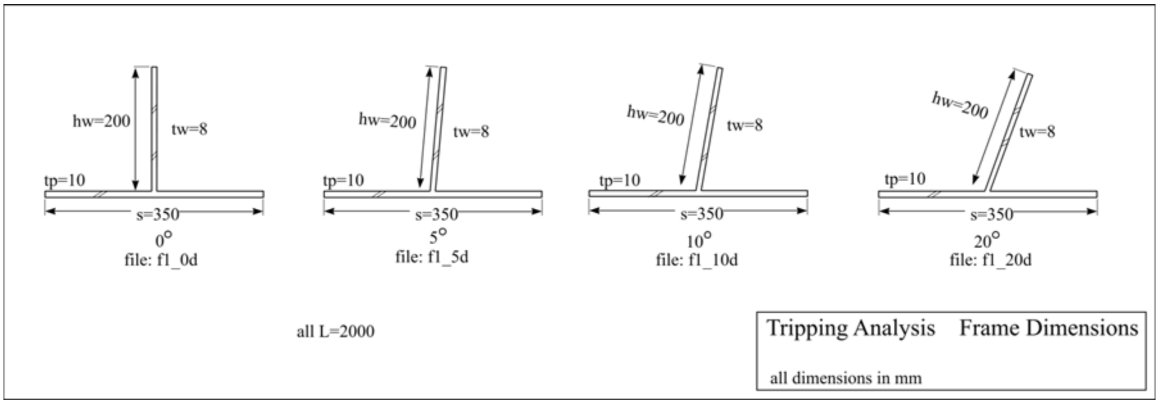

Figure 9. Flat bar sections analyzed.

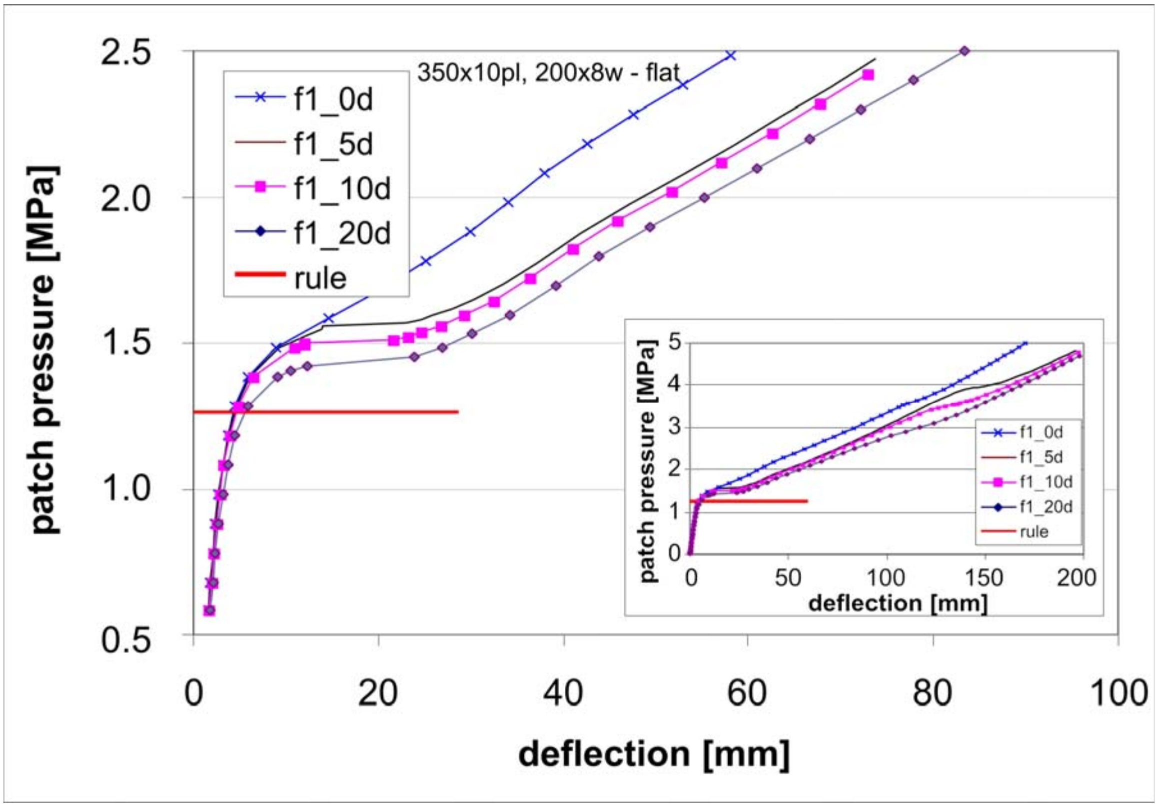

Figure 10. Load vs. deflection for flat bar sections.

The above Figures illustrate several notable results. First it is clear that the rule equation is valid for all twelve cases shown. Neither the type of section, not moderate tilt angles affects the validity of the rule values.

For the tee and flat bar sections, the load deflection curves typically exhibit a small ‘jump’. This behavior could be termed a ‘local instability’ (these are the short horizontal segments in the pressure-deflection curves). There is a temporary loss of stiffness. The load capacity continues to rise after these interruptions. All these have occurred above the design load, and only represent a minor effect. This behavior is not seen in the angle sections, which tend to respond more progressively.

The final key observation is that all sections exhibit a substantial reserve capacity, even though the sections are tending to fold over. Figure 5 along with Figure 4 illustrate this for angle sections. Note that the capacity continues to rise as the deflection (of the shell) increases. At the maximum deflection the load is also at maximum. At 6 MPa the frame is folded over, without a loss of capacity. This can only be due to the membrane (tensile) stresses in both the shell plate and frame. The membrane behavior is providing the substantial reserve. Similar behavior occurs in all sections analyzed. As a result, there is no need to impose a tripping requirement in the PUR.

3. Local Buckling Analysis

The frames listed in Table 4 were analyzed to check the local buckling requirements. The frames are identical in all aspects except for the web height. The results illustrate the effect of the web slenderness on the load-deflection curve. Figure 11 shows the values for four frames. The tallest of the frames has a web height of 372 mm and thickness of 8 mm. This frame experiences local web just above the design point, and has almost no reserve strength, and a drastic drop in capacity after buckling. The next frame, with a 350 mm web also shows a drop in capacity, but is able to sustain much of the load after buckling. The next frame with a 300 mm web is able to hold the design load after local buckling, but no additional load. The fourth frame, with a 200 mm web has a significant amount of post collapse reserve strength.

The same data is plotted in non-dimensional form in Figure 12. It is clear that the shorter webs have relatively more reserve. The 372 web just meets the web slenderness ratio of $805/\sqrt{fy}$. This shows that the 805 value is just sufficient to ensure the design capacity, but does not ensure any reserve. Figure 12 shows that a web slenderness value of $<650/\sqrt{fy}$ would be needed to ensure a plastic reserve.

It is noteworthy that tee sections at the rule limit of web slenderness have no plastic reserve, and yet flat bar sections do. The data shown in Figure 10, which are for flat bars with a web slenderness of $433/\sqrt{fy}$. The well exceeds the rule value for flat bars of $282/\sqrt{fy}$. It is possible to see that the web has experienced some local buckling near the supports. These tend to be small and do not effect the reserve capacity of the frame.

Table 4. Properties of Tee Frames Analyzed for Local Buckling

Item [units]	**value(s)**	**value(s)**	**value(s)**	**value(s)**
file	t1_0d	t1_AL5	t1_AL3	t1_AL4
frame type	angle	Angle	angle	angle
frame length [mm]	L=2000	L=2000	L=2000	L=2000
load length [mm]	b=500	b=500	b=500	b=500
web height [mm]	**hw=200**	**hw=300**	**hw=350**	**hw=372**
flange width [mm]	bf=75	bf=75	bf=75	bf=75
frame spacing [mm]	s=350	s=350	s=350	s=350
web thickness [mm]	tw=8	tw=8	tw=8	tw=8
plate thickness [mm]	tp=10	tp=10	tp=10	tp=10
flange thickness [mm]	tf=10	tf=10	tf=10	tf=10
Elastic Modulus [MPa]	E=207000	E=207000	E=207000	E=207000
yield strength [MPa]	fy=300	fy=300	fy=300	fy=300
Tangent Modulus [MPa]	Et=500	Et=500	Et=500	Et=500
inclination [deg.]	**phi=0**	phi=0	phi=0	phi=0

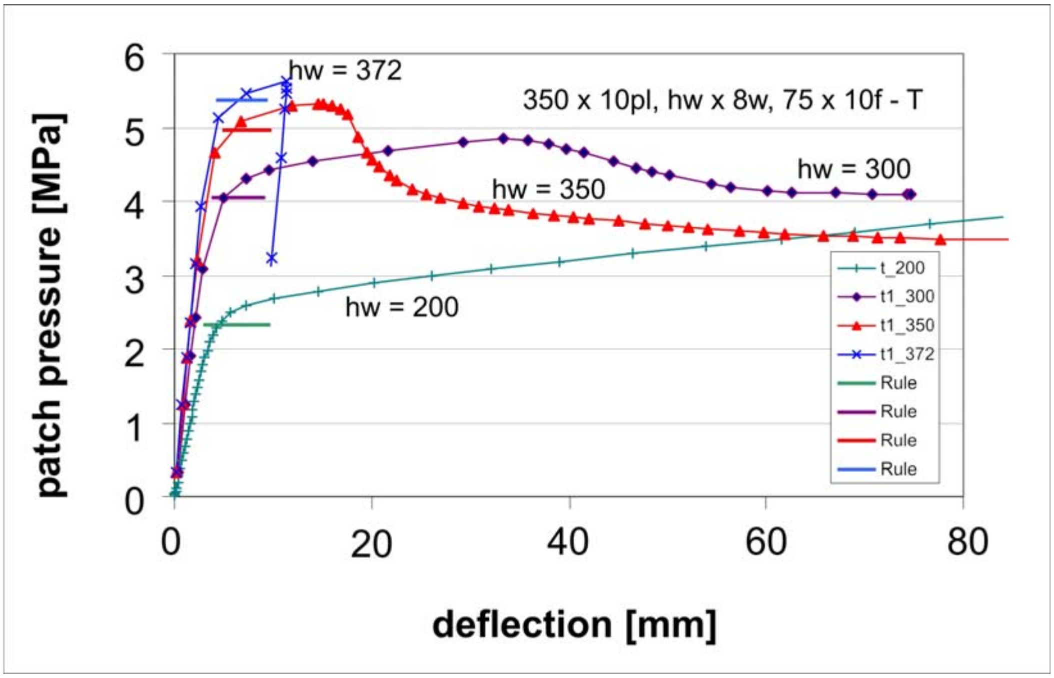

Figure 11. Load vs. deflection for tee sections with various web heights.

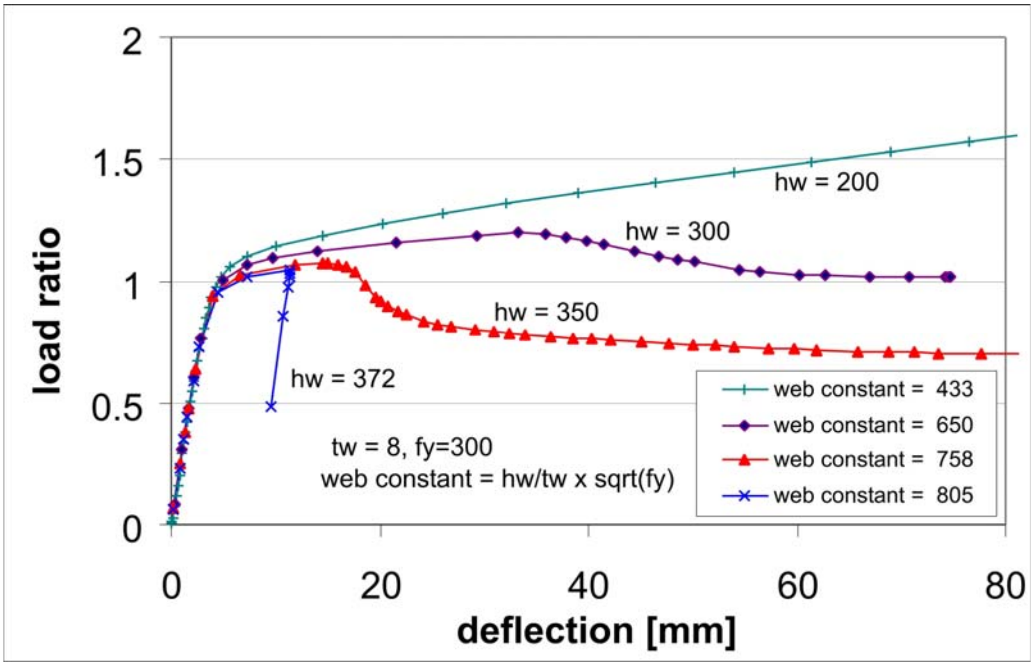

Figure 12. Load ratio (load/rule value) vs. deflection for tee sections with various web heights.

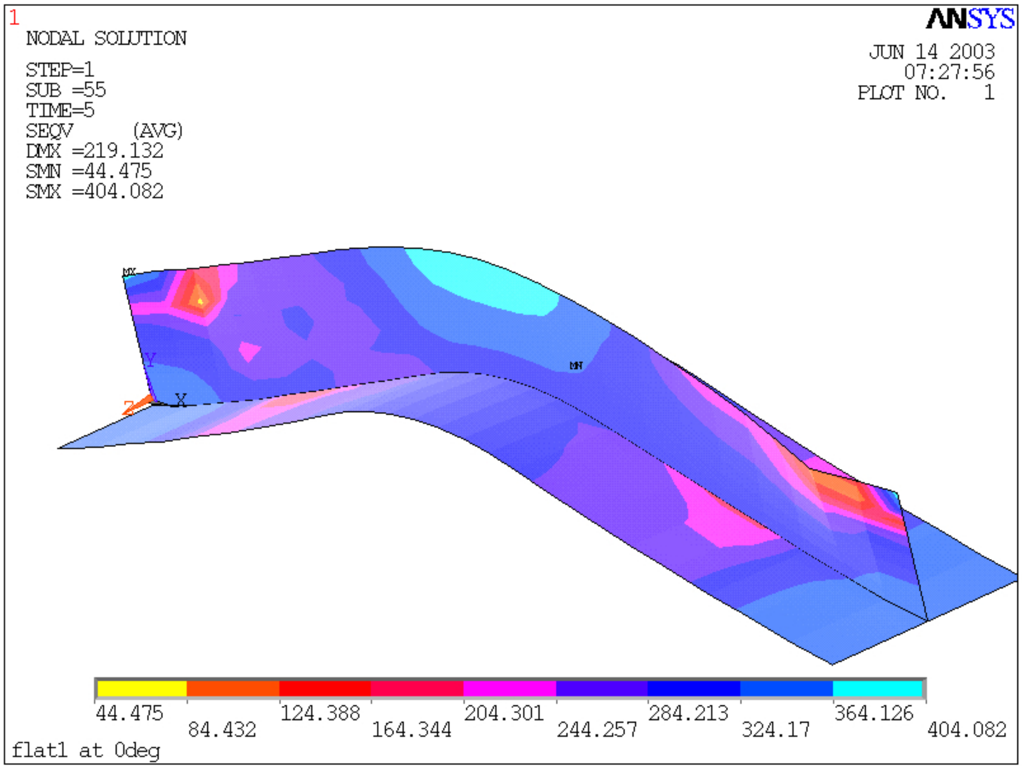

Figure 13. Stress and deflected shape of the flat bar frames shown in Figure 9, under a load of 4x the design load.

4. Conclusions

The above analysis suggests that tripping is not a problem for the types of frames and loading considered in the PUR. While the webs do tend to fold at large deflections, there is no significant loss of capacity. Actually, there appears to be a significant plastic reserve of strength at large deflections, as long as the frame does not experience local buckling.

Local buckling is controlled by web slenderness equations. The equation for tee and angle sections is $hw/tw \leq 805/\sqrt{fy}$. This appears to be just sufficient to ensure that the design load can be supported. However, it appears that at such a slenderness, there is no plastic reserve. It may be appropriate to adjust the rule to something like $hw/tw \leq 650/\sqrt{fy}$ for tees and angles, to ensure that the frames have a plastic reserve at large deflections.

The opposite case appears to exist for flat bars. The current local buckling rule for flat bars is $hw/tw \leq 282/\sqrt{fy}$. The analysis has shown no significant problems even for cases in which $hw/tw = 433/\sqrt{fy}$. There would likely be significant practical advantages if flat bar frames could be made taller than currently allowed. This is an area where more work is recommended before a suggestion for a rule change can be made.

References

ANSYS 6.0, Finite Element Program by SAS IP, Inc, 2001 (‘University High’ Version)

Annex 1 – ANSYS Listing for typical analysis.

```
/title,a1_5d

/prep7

!define variables
!units: mm,MPa
L=2000
hw=200
bf=75
s=350
tw=8
tp=10
tf=10
E=200000
Et=500
fy=300
ang=5
cos=1-.000152*ang*ang
sin=.0173*ang

n1=8
n2=2

!element types
et,1,shell181
ex,1,E
nuxy,1,0.3
tb,bkin,1
tbdata,1,fy,Et

r,1,tp


et,2,shell181
r,2,tw

et,3,shell181
r,3,tf



!keypoints

n=0
lx=0
k,n+1,lx,-tp/2,-s/2
k,n+2,lx,-tp/2,0
k,n+3,lx,-tp/2,s/2
k,n+4,lx,(hw+tf/2)*cos,(hw+tf/2)*sin
k,n+5,lx,(hw+tf/2)*cos-bf*sin,(hw+tf/2)*sin+bf*cos
n=5
lx=750
k,n+1,lx,-tp/2,-s/2
```

```
k,n+2,lx,-tp/2,0
k,n+3,lx,-tp/2,s/2
k,n+4,lx,(hw+tf/2)*cos,(hw+tf/2)*sin
k,n+5,lx,(hw+tf/2)*cos-bf*sin,(hw+tf/2)*sin+bf*cos
n=10
lx=1250
k,n+1,lx,-tp/2,-s/2
k,n+2,lx,-tp/2,0
k,n+3,lx,-tp/2,s/2
k,n+4,lx,(hw+tf/2)*cos,(hw+tf/2)*sin
k,n+5,lx,(hw+tf/2)*cos-bf*sin,(hw+tf/2)*sin+bf*cos
n=15
lx=2000
k,n+1,lx,-tp/2,-s/2
k,n+2,lx,-tp/2,0
k,n+3,lx,-tp/2,s/2
k,n+4,lx,(hw+tf/2)*cos,(hw+tf/2)*sin
k,n+5,lx,(hw+tf/2)*cos-bf*sin,(hw+tf/2)*sin+bf*cos

!define lines conecting keypoints
!sizing lines (for mesh density)

n=0
m=0
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lesize,m+1,,,n1
l,n+2,n+3
lesize,m+2,,,n1
l,n+2,n+4
lesize,m+3,,,n1
l,n+4,n+5
lesize,m+4,,,n2

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lesize,m+5,,,n1
l,n+2,n+7
lesize,m+6,,,n1
l,n+3,n+8
lesize,m+7,,,n1
l,n+4,n+9
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lesize,m+9,,,n1

n=5
m=9
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l,n+2,n+4
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l,n+4,n+5
lesize,m+4,,,n2

l,n+1,n+6
```

```
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l,n+5,n+10
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m=27
l,n+1,n+2
lesize,m+1,,,n1
l,n+2,n+3
lesize,m+2,,,n1
l,n+2,n+4
lesize,m+3,,,n1
l,n+4,n+5
lesize,m+4,,,n2

!create area
n=0
a,n+1,n+2,n+7,n+6 !1
a,n+2,n+3,n+8,n+7 !2
a,n+2,n+7,n+9,n+4 !3
a,n+4,n+5,n+10,n+9 !4

n=5
a,n+1,n+2,n+7,n+6 !5
a,n+2,n+3,n+8,n+7 !6
a,n+2,n+7,n+9,n+4 !7
a,n+4,n+5,n+10,n+9 !8
n=10
a,n+1,n+2,n+7,n+6 !9
```

```
a,n+2,n+3,n+8,n+7 !10
a,n+2,n+7,n+9,n+4 !11
a,n+4,n+5,n+10,n+9 !12

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asel,a,area,,9,10
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AATT,1,2,,

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asel,all

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nsel,all

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d,all,uz,0
nsel,all


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nsel,all

nsel,r,loc,x,0
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nsel,all
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cnvtol,M,,0.01,,1
ncnv
pred,on,,on
outres,all,all


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ftran
ksel,all

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deltim,0.02,0.01,0.1
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sfa,all,,pres,py1
sftran
asel,all

save
solve
save
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