STUDY ON BEHAVIOUR OF EFFICIENT COLD FORMED OPEN CROSS SECTION FOR RACK COLUMN

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ABSTRACT

The aim of this paper is to choose the efficient cold formed steel section profile for a fixed cross-sectional length, thickness and height of the column. In this work six types of sections wereselectedsuchasRC-1600-1.6-SP1,RC-1600-1.6-SP2,RC-1600-1.6-SP3,RC-1600-1.6-SP4,RC-1600-1.6-SP5 and RC-1600-1.6SP6 .The efficient section is chosen based on the design buckling resistance of the column by performing Numerical and Theoretical analysis.

The Numerical analysis is performed using ANSYS 12.The elastic section properties of thecolumnsarepredictedbyperformingFinitestripanalysisbyusingopensourceprogramCUFSM.The design buckling resistance of the columns are predicted by using Finite element toolANSYS.The columns are chosen with a cross sectional length of 544.36 mm and a thickness of 1.6mm. The end condition of the column is considered as hinged-hinged. The height of the column is1600mm.Thematerialpropertiesof the cold formed steel such as Yield Stress, Modulus of Elasticity, Tangent Modulus,etc are obtained by conducting the coupon test.

The theoretical analysis is performed using various codal provisions such as Direct Strength Method - S100:2007 (North American AISI Specifications), Australian/New Zealand Standard Code - AS/NZ: 4600- 2005 and by British Standard BS EN 1993-1-1-3: 2006.The bilinear elastic-perfect plastic behavior of material is considered for analysis. The material and geometric nonlinearity is included in the finite element model. The linear elastic buckling analysis followed by non-linear ultimate strength analysis is performed to obtain the buckling load, buckling mode and ultimate load capacity of the sections. In theoretical analysis the Design buckling resistance may only vary based on the reduction factor and effective cross sectional area of the column.

The buckling resistance of the column obtained from both Numerical and Theoretical analysis are compared for all types of cross section and thus the efficient cross section is selected.

KEYWORDS:*CUFSM,ElasticBuckling,AS/NZ,BucklingResistance,YieldStress,ModulusofElasticity,TangentModulus.*

1.0.INTRODUCTION

Cold-formedsteelstructuralmembersmay lead to a more economic design than hot-rolled members as a result of their high strengthtoweightratio,easeofconstructionandsuit ability for a wide range of applications. Thethickness of steelsheetsor strip generally usedin cold-formedsteel structural members rangesfrom 0.0149inch (0.4mm) to about 0.25 inch(6.4 mm). Steel plates and bars as thick as 1 in(25 mm) can be cold-formed successfully intostructuralshapes. Thematerialsusedincold-formedthin-wallmembershavetosatisfycertaincriteriatobesuit ableforcold-formingandusually also for galvanizing. The yield strengthis normally in the range of 220 to 350 N/mm^2,butalsosomehigh-strengthsheetsteelswithyield strength of over 500 N/mm^2 are used insomecasesduetopracticalreasonsi.e.transportati on, handlingetc., limit the range ofthicknessofthematerialused inprofileds heeting.Coldforminghastheeffectofincreasingthe yieldstrengthofsteel,theincreasebeing the consequence of cold working well intothe strain-hardening range. These increases arepredominant in zones where the metal is bent byfolding. The effect of cold working is thus toenhance the mean yield stress by 15% - 30%.For purposes of design, the yield stress may beregardedashavingbeenenhancedbyaminimumo f15%.Thedepthofcold-formedindividualframingmembersrangesfrom51t o 305mm and the thickness ofmaterialrangesfrom1.2to6.4mm.Insomecases,t hedepthof members may be up to 450 mm andthe thickness of the member may be 13 mm forbuildingconstruction.Cold-formedsteelplatesections in thickness of up to about 19 or 25 mmhavebeenusedinsteelplatestructures,transmissionpole s,andhighway-signsupportstructures.

The following are the buckling modes ofcoldformedsteelsections,

1. LocalBuckling
2. DistortionalBuckling
3. Euler'sbuckling(Flexuralorflexural –torsional)

1.1.LocalBuckling

TheplateelementsofCold–formedsectionsarenormallythinhigherplateslende rness ratio and hence they buckle locallybefore yieldstress is reached,Localbucklingmode of a given thin – walled member depends,onitsi)crosssectiongeometry(shape&di mensions) and ii) and support conditions. Theelastic local buckling of thin elements does notimmediately lead to failure. The elements cancarryadditionalloadinthepost–bucklingstrengthbeforefailureoccurs.ThePost–buckling strength of elements having relativelylarge flat width to thickness ratio may be severaltimestheloadthatcauseslocalbuckling.Con sequentlyallthecold–formeddesignspecificationstakeintoaccountthepo st–buckling strength. The figure 1.1 shows the localbucklingfailure

1.2. FlexuralBuckling

Inthismodecompressionbucklesoutweakerprinci pleaxis&collapsesoccuratrate Individual

Fig.1.1-LocalBuckling

Fig.1.2-FlexuralBuckling

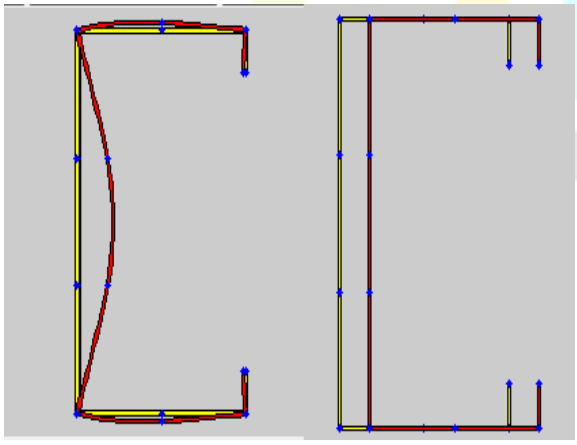
Following excessive buckling deformation (notwisting).Normally,Longcolumnswillunderg oflexural buckling along half wave lengths. Thefigure1.2showsthepureflexuralbuckling

1.3. TorsionalBuckling

IntheTorsionalbucklingmode,themembers fail by twisting about the longitudinalaxisthroughtheShearcentre(Nobending)Flexural Torsional Buckling. Due to the smallerthickness the section have low torsional stiffnessandtheirshearcentreandcentroidare locatedawayfromeachother.Thiscausesflexural–torsionalbuckling(simultaneousbendingandtwistin g).Figure1.4showstorsionalbuckling

1.4. DistortionalBuckling

Distortionalbuckling,alsoknownas“Stiffene r buckling” or “Local torsional buckling”is mode characterized by a rotation of the flange attheflangeWebjunctioninnumberswithedgestiffen ed elements. In members with intermediatestiffenedelementsdistortionalbuckling ischaracterized by displacement of the intermediatestiffenernormaltotheplaneofthe element.Figure 1.3showsthedistortionalbuckling.



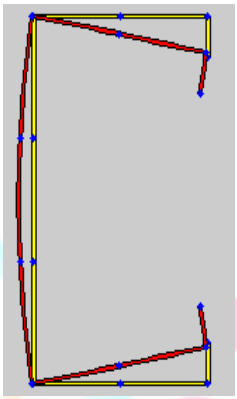


Fig.1.3-Distortional

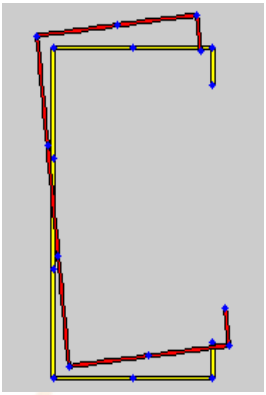


Fig.1.4-Torsional

2.0. General

In this work, six cross sectional profilesare chosen based on the stiffener, flange and lip.Thedimensionsofthecrosssectionarefinalized basedontheguidelinesgiveninvarious codal provisions on maximum allowableflat-width ratio. The mode of failure in all thecasesistorestrictlengthanddistortionalbucklin g of the specimen using CUFSM. Thethicknessofthesectionsis1.6mmandthelength of thespecimen is 1600mm.The detailsofthespecimenaretabulatedin1.0

RC	Rack column
1600	Heightofthespecimen
1.6	Thicknessofthespecimen
SP1	Sectionprofilenumber

Table.1.0-Labelligofspecimen

2.1. MaterialProperties

Description	Values
Modulusofelasticity	$2X10^5N/mm^2$
Poisson'sratio	0.3
FinalTangentmodulus	$2X10^4N/mm^2$
Yieldstressofthematerial	$248N/mm^2$
Crosssectionallength	544.36mm
Thicknessofthematerial	1.6mm
Heightofthecolumn	1600mm
Endcondition	Hinged-Hinged

Theassumedmaterialpropertiesaretabulatedin1.1

Table.1.1-MaterialProperties

3.0.Sectionprofiles

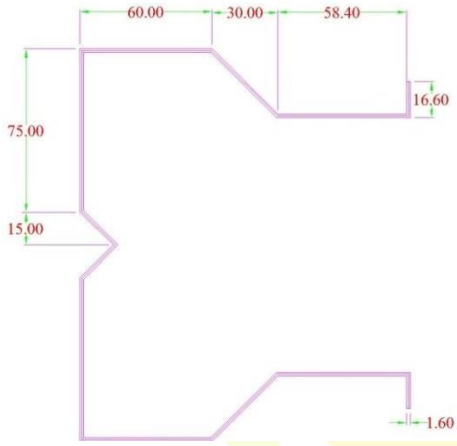


Fig.1.5-RC-1600-1.6-SP1

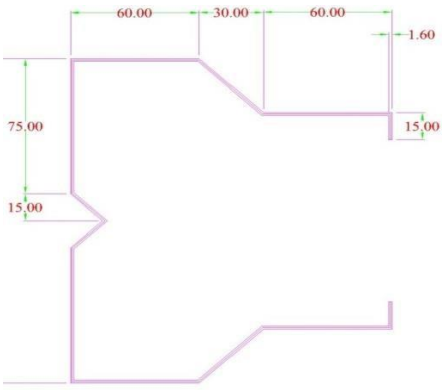


Fig.1.6-RC-1600-1.6-SP2

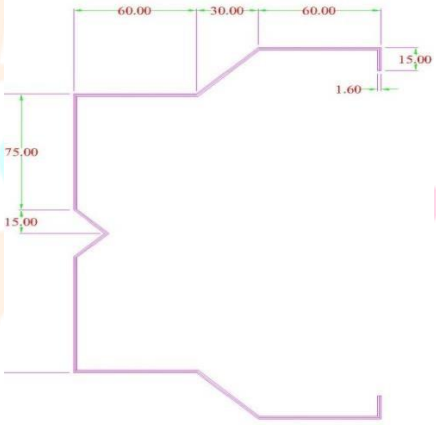


Fig.1.7-RC-1600-1.6-SP3

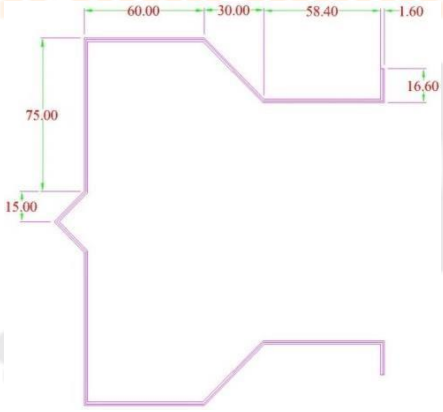


Fig.1.8-RC-1600-1.6-SP4

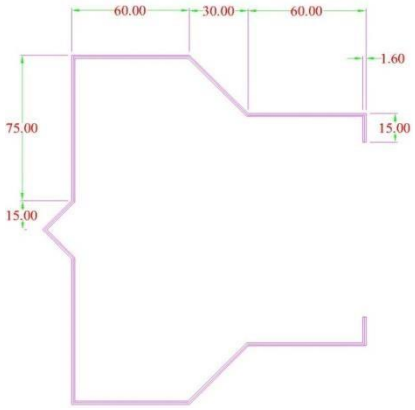


Fig.1.9-RC-1600-1.6-SP5

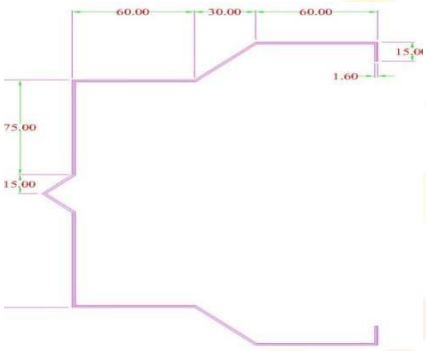


Fig.2.0-RC-1600-1.6-SP6

TheElementID suchas A,L,C,J,FandG arechangeable similarly B, K, D, I, E and H areunchangeablearetabulatedin1.2.

	ElementID		
SpecimenID	A,L	C,J	F,G
RC-1600-1.6-SP1	O	I	I
RC-1600-1.6-SP2	I	I	I
RC-1600-1.6-SP3	I	O	I
RC-1600-1.6-Sp4	O	I	O
RC-1600-1.6-SP5	I	I	O
RC-1600-1.6-SP6	I	O	O

I=Inwardbend,O=Outwardbend

Table.1.2-Inwardbend&outwardbend

4.0.NumericalAnalysis:

ThenumericalanalysiswasperformedusingANSYS 12.0topredictthedesignresistanceoftherack column. The columns were analyzed using 4nodedshellelementwithendconditionasHinged-Hinged. The sharp edge radius are not consideredbecause the radius less than the 5 times thicknessof the specimen (r<5t) and less than the 0.10 timesthe flat width ($r<0.10b_p$). A bilinear elastic-perfectplastic behavior for material was considered. Thematerial and geometric nonlinearity was includedinthefiniteelementmodel.Thelinearelasticbucklinganalysiswasperformedtoobtainthebuckling loads and bucklingshapes of the rackcolumn. The nonlinear analysis was performed toobtain the design resistance of the column. Therotationisrestrainedattopandbottomofthecolumn. The column is meshed and divided in to anumberofelementsasfiguredin2.0&2.1.

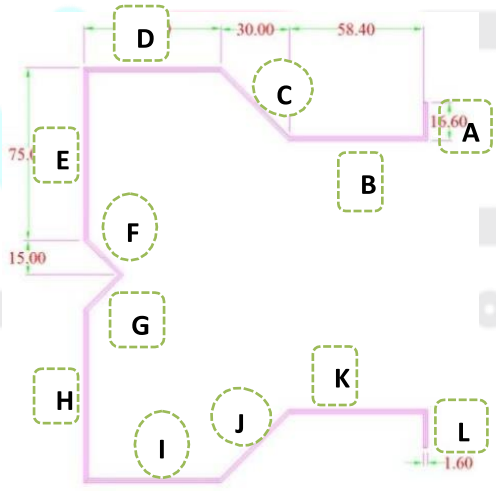


Fig.2.1-Elementsnotation

ElementID	No.ofdivisions
A,L	2
B,K	6
C,J	5
D,I	4
E,H	4
F,G	2

TTabe.1.3-
No.ofdivisionsofElementsformeshing

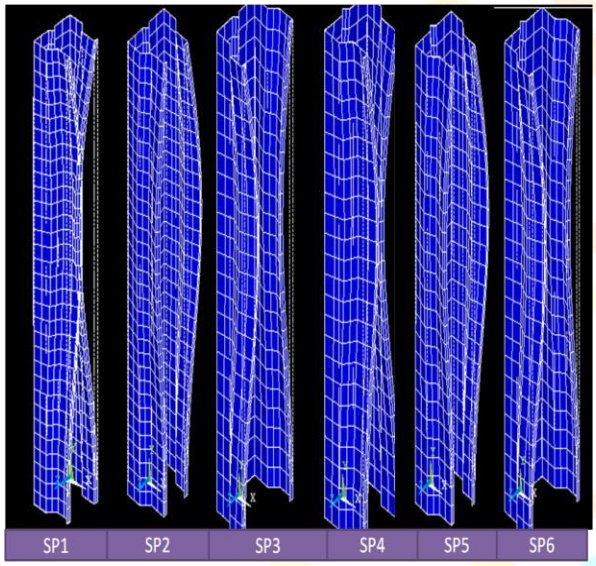
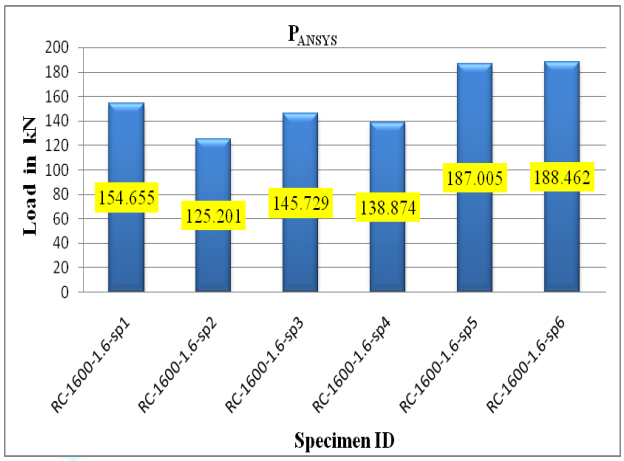


Fig.2.2-Meshingofdifferentspecimens

TheNon-linearFiniteElementAnalysisisperformed for all the specimens and the highestvalueis188.462kNforRC-1600-1.6-SP6specimen. The results are forsix specimens arecomparedin Fig.2.2



Fi
g.2.3-Loadcarryingcapacity(P_{ANSYS})

5.0.TheoreticalAnalysis

Thetheoreticalstudyinvolvestheanalysisof various shapes of cold formed steel column toobtainitsloadcarryingcapacity.Theloadcarryin g capacity shall be obtained using Eurocode(BSEN1993-1-1-3:2006)andDirectStrengthMethodbyNorthAmer icanSpecifications(AISIS100-2007).Thedetailssuch as load factor and length of the specimenare took from the CUFSM and the load carryingcapacity is derived. The lengths obtained fromCUFSMarerestricteduptodistortionalbuckli ng.Thevalueofdistortional bucklingload factor for all the specimens are less thanthelocalbucklingloadfactorandhencethedisto rtional buckling occurs. The load carryingcapacity for all the specimens is compared inFig.2.3&2.4.

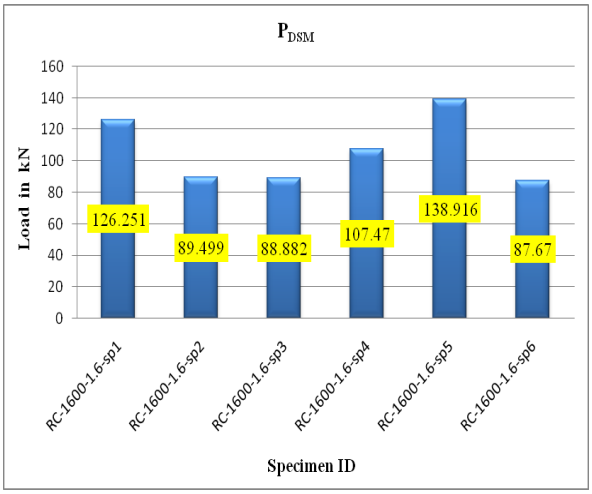


Fig.2.4-Loadcarryingcapacity(P_{DSM})

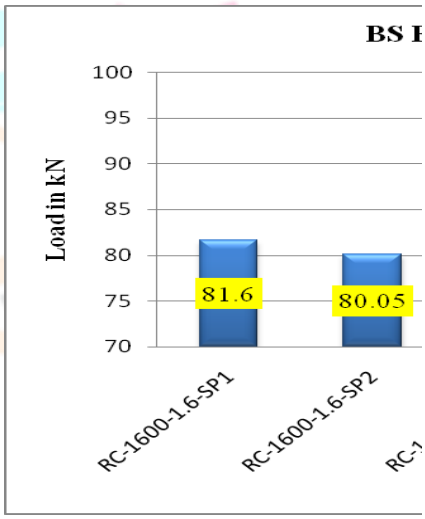


Fig.2.5-Loadcarryingcapacity(P_{EN})

6.0.Comparisonofresults

TheAnsysandEurocode(BSEN1993-1-1-3:2006)comparedtoDSMresultsasshownintheFig.2.5.TheresultsshowtheEurocodeislessthantheDSMresult.

SpecimenID	NumericalResults(kN)	Theoreticalresults	
		P_{EN}(kN)	P_{DSM}(kN)
RC-1600-1.6-SP1	154.655	81.6	126.251
RC-1600-1.6-SP2	125.201	80.50	89.499
RC-1600-1.6-SP3	145.729	92.766	88.882
RC-1600-1.6-SP4	138.874	94.898	107.47
RC-1600-1.6-SP5	187.005	82.558	138.916
RC-1600-1.6-SP6	188.462	92.558	87.67

Table6.1-loadcarryingofdifferentspecimenbyNumerical&theoretical

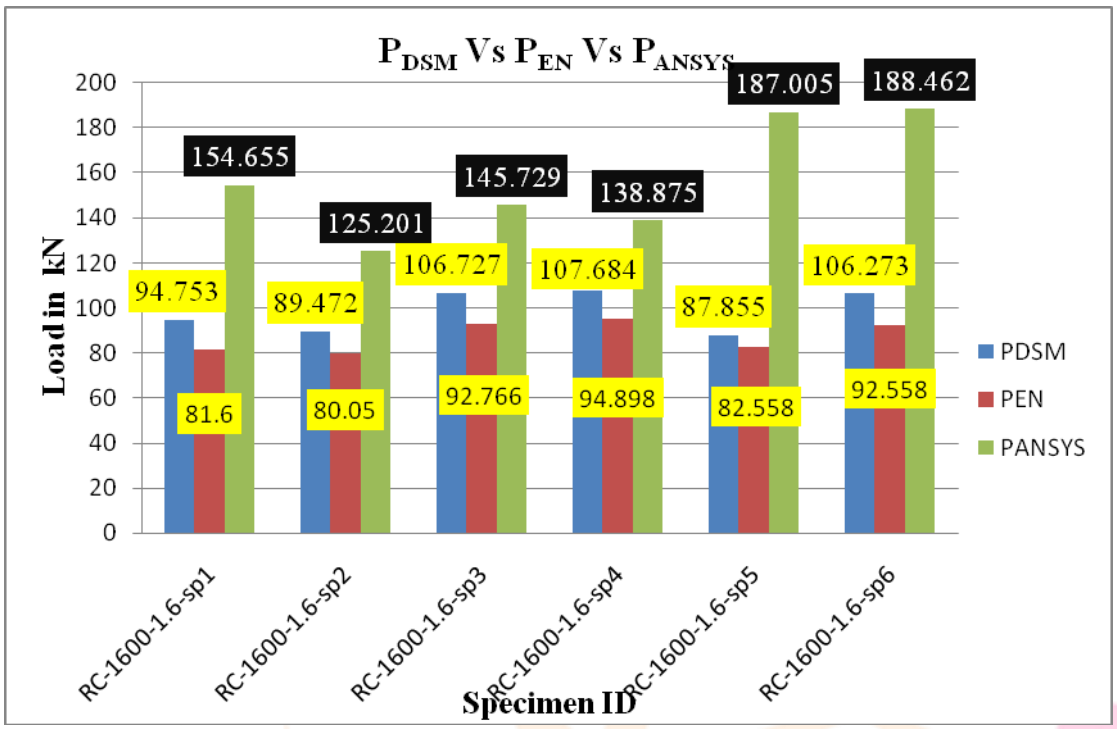


Fig6.2-Comparison of P_{DSM}, P_{EN}, P_{ANSYS}

6.4.COMPARISONOFTHEORETICALANDNUMERICALANALYSIS:

TheComparisonofboththeoreticalandnumericalresultsobtainedisshowndiagrammatically belowinFig.6.3.

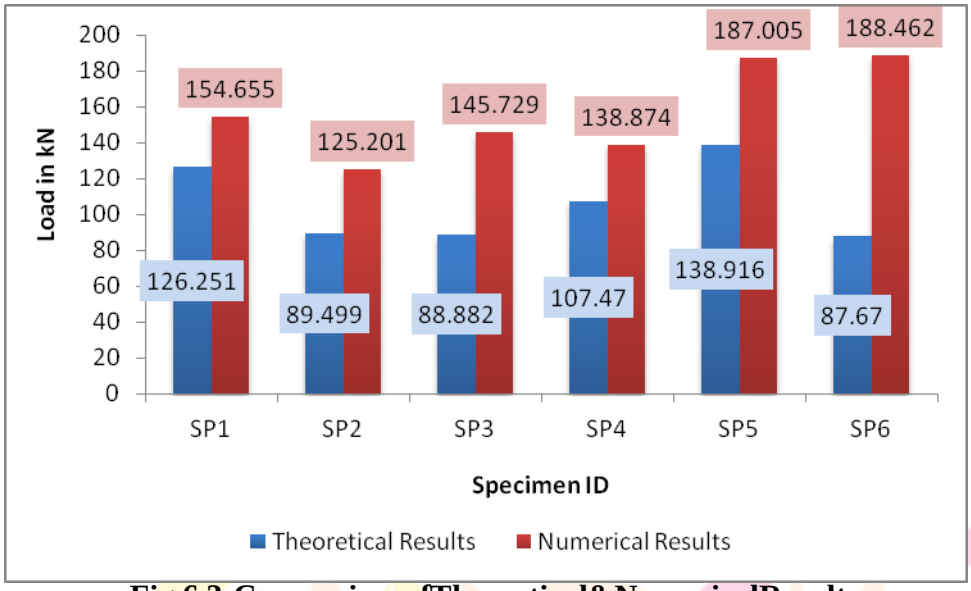


Fig.6.3-ComparisonofTheoretical&NumericalResults

7.Conclusions

Fromtheaboveresults,optimumsectionprofilecannotbepredictedatthisstageduetothefollowingreasons

- Wecancometotheconclusiononlyafterperformingalltheanalysis(bothnumerical&theoretical)andcomparingitwiththeexperimentalresults.
- Also, the optimum solution cannot be predicted either with the results obtained fromFinite Element Software at this stage. The behaviour of the column in experimentalinvestigationhastobestudiedanda goodsimulationtotheexperimentalresultshastobeachieved byoptimizingthemeshsize.

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