A Study on the Effect of Adjacent Segments of a Steel I-Beam with Lateral Supports on Its Critical Moment

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Abstract. One of the most common failure modes in a steel beam structure is lateral torsional buckling (LTB). The unbraced length of the beam is a crucial factor for LTB analysis. The current formula to calculate the critical moment with respect to LTB analysis has not considered the effect of adjacent segments of steel beams with lateral supports. The adjacent segments of beams actually provide restraints to the torsional rotation while maintaining the possibility of warping to occur. This research was intended to observe the effect of the adjacent segments on its critical moment by using the linear buckling analysis and collapse analysis of a finite-element model. The results from the FEM analysis were compared to the value of the critical moment obtained from the available formula in the standard. The linearised buckling analysis results show an increase in the critical moment by 7.02% to 154.83%, while the collapse analysis results show an increase of 2% to 40.41%. As the adjacent segments becomes longer, the increase is less due to the yielding of adjacent beam segments, contributing less stiffness to the beam spanning in between lateral supports.

1 Introduction

Recently, steel has become the main choice of construction material due to its strength and efficiency. Steel is considered environmentally friendly because it generates a smaller amount of waste compared to concrete, and steel can also be reused and recycled. However, the current formula used to design various structural elements using steel has not accounted for many possible cases, which is worth looking into.

A beam is a major structural element that resists loads applied laterally its axis. It is designed to resist shear forces and bending moments resulting from the loading. Steel beams are primarily designed as flexural members while shears are usually not a controlling factor in the design of a beam.

There are several possible failure modes of a beam, i.e. reaching its plastic moment M_p and/or by buckling. The plastic moment of a steel beam is reached when the bending stress everywhere in the section reaches yielding stress.

Buckling in a beam can occur as lateral torsional buckling (LTB) or as local buckling in the flange or web of a steel cross section. This research focuses on the LTB case which is considered to be the most common failure mode in a steel beam.

LTB is a phenomenon in which the beam tends to twist and displace laterally. It is an overall effect in which the compressive flange, which behaves like an axially loaded member, tends to buckle about its strong axis in a lateral direction. However, the web and tensile flange resist this tendency to a certain extent. LTB in a beam can be prevented by providing lateral supports at intermediate points in a beam or by using a torsionally strong steel section such as a box or a tube section.

As stated previously, providing lateral supports or in other words controlling the unbraced length may prevent LTB. Beams with longer unbraced length tend to buckle more easily and vice versa. Lateral supports limit the unbraced length of the beam and thus increase its critical moment.

The study of lateral torsional buckling has been conducted by many researchers. Wijaya (2011) [3] concluded that by taking into account adjacent panels, the elastic critical load by finite element modeling using SAP is increased 1,5 to 2 times compared to the critical load calculated considering only as a single span of a laterally braced beam. Wijaya (2012) [4] compared the value of the critical moment while considering the beam with lateral support as a whole. The critical moment obtained from the solution of a differential equation using the finite difference method was found to be higher than the value in AISC (2010) [1]. Wijaya (2013) [5] mentioned that the adjacent segment gave significant restraint and thus should not be ignored. This research proved that the effective length method (ELM) for lateral torsional buckling analysis (Ziemian (2010) [6]) is conservative when the adjacent beam is stiffer. However, ELM for a concentrated load was not found to be under-estimated and unsafe for use while for the uniform end-moment case, ELM values were acceptable (Wijaya (2013) [7]).

Nguyen, et al (2013) [8] have conducted research on the influence of the boundary condition and geometric

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imperfections on lateral torsional buckling resistance of a fibre-reinforced I-beam by finite element analysis. Sanchez (2012) [9] has drawn a comparison between lateral torsional buckling on beams under different boundary conditions, i.e. fixed on both ends and simply supported.

The current formula in AISC (2016) [2] to calculate the critical moment with respect to LTB analysis has not considered the effect of adjacent segments of steel beams with lateral supports. The adjacent segments of beams actually provide restraints to the torsional rotation while maintaining the possibility of warping to occur. Previous research has suggested that the adjacent segments give significant restraints to the beam in between lateral supports. The beam has to be analysed as a whole in order to obtain the value of its critical moment. This research was intended to observe the effects of the adjacent segments to the critical moment of steel I-beams in between lateral supports by using the linear buckling analysis and the collapse analysis of a finite-element model. The collapse analysis was carried out as a better representation of critical moment estimation by using finite element analysis models.

2 Lateral Torsional Buckling

AISC [2] classifies LTB into elastic and inelastic LTB. If the maximum bending stress is less than the proportional limit when buckling occurs, the failure is elastic. Otherwise, it is inelastic.

The nominal flexural strength M_n is calculated depending on the buckling region which is based on the unbraced length of the beam section L_b, as shown in Figure 1.

If L_b, a distance between points braced against lateral displacement of compression flange is less than the limiting laterally unbraced length for the limit state of yielding, L_p, the plastic strength M_p controls and no LTB occurs. However, if L_b is more than the limiting laterally unbraced length for the limit state of inelastic LTB L_r, elastic LTB occurs. When $L_p<L_b\leq L_r$, inelastic LTB occurs.

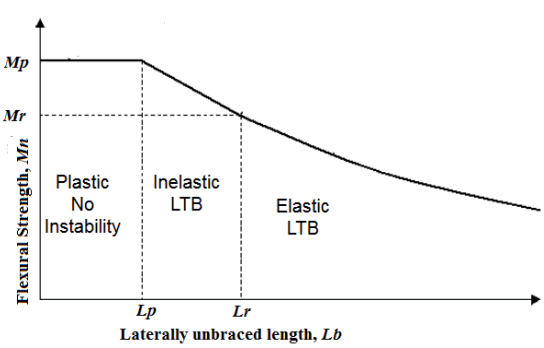

Fig. 1. Nominal flexural strength diagram based on the laterally unbraced length

AISC [1] has formulated the value of L_p and L_r for doubly symmetric compact I-shaped members and channels as following:

$$L_p = 1.76\, r_y \sqrt{\left(\frac{E}{F_y}\right)} \tag{1}$$

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \tag{2}$$

where E is the modulus elasticity of steel, F_y is the specified minimum yield stress, r_y is the radius of gyration about y-axis, J is the torsional constant, c is taken as 1 for I-shaped members, S_x is the elastic section modulus about x-axis, and h_o is the distance between flange centroid.

AISC [1] also has formulated the flexural strength of compact I-shaped and channel members as stated in Equations 3 to 6. Plastic moment M_p is defined as:

$$M_p = Z_x F_y \tag{3}$$

where Z_x is the plastic section modulus about the x-axis.

For inelastic LTB ($L_p<L_b\leq L_r$), the flexural strength M_n is calculated as:

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \tag{4}$$

where C_b is the lateral torsional buckling modification factor for the non-uniform moment.

For elastic LTB ($L_b>L_r$), the flexural strength M_n is calculated as:

$$M_n = M_{cr} = F_{cr} S_x \leq M_p \tag{5}$$

where F_{cr} is the critical stress taken as:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{6}$$

From all the equations above, it can be discerned that the effect on adjacent segments of a steel beam is not accounted for. The current formula to calculate the flexural strength of the I-beam is using the assumption that at both ends of a laterally restrained beam, the torsional rotation is restrained but warping is still free to occur. As a matter of fact, the presence of adjacent segments provides partial restraint to the warping of the beam spanning in between lateral supports. Thus, this research observed the effects of the length and different cross-sections of adjacent segments on the critical moment of the mentioned beam.

3 Finite Element Modelling

3.1 Verification Models

Finite element modeling was conducted using the aid of a computer program called ADINA v9.3.2 by using both the linearized buckling analysis and the collapse analysis.

The collapse analysis was carried out by applying an incremental load on the beam structures until collapse.

For the buckling analysis, the analysis was conducted by introducing initial geometrical imperfection on the beam. The distribution of the geometric imperfection was taken from the first buckling mode of the linearized buckling analysis using Eigen value analysis which was carried out prior to the collapse analysis, where the amplitude of imperfection was taken to be L_b/4000. The yield strength f_y of the steel I-section was taken to be 250 MPa.

There were two steel sections being modeled in this research, i.e. Model A (IWF350x175x7x11) and Model B (IWF400x200x8x13). The length of beam spanning in between lateral support was modeled as L_b=4m and L_b=7m while the length of adjacent beam section L_s varied between 2m, 3m, 4m, and 5m. Figure 2 and Table 1 shows the configuration and variation of span in each model.

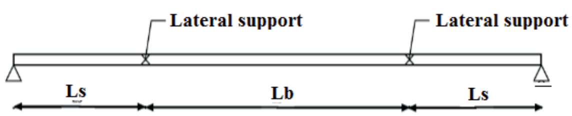

Fig. 2. Steel beam with lateral supports being modeled

Table 1. Variation of L_b and L_s being analyzed

Model No.	L_b (m)	L_s (m)	Total span length, L (m)
1A, 1B	4	2	8
2A, 2B	4	3	10
3A, 3B	4	4	12
4A, 4B	4	5	14
5A, 5B	7	2	11
6A, 6B	7	3	13
7A, 7B	7	4	15
8A, 8B	7	5	17

To observe the effect on the adjacent segments, the beam models were applied with three loading cases: (1) a uniformly distributed load of across beam span, applied at each node along the longitudinal line where the shear centre lies, (2) a concentrated load at midspan also applied the shear centre of the beam cross-section, and (3) a uniform end-moment formed by equal force in opposite direction applied at top flange and bottom flange in opposite direction.

The boundary condition at both ends of the beam was modeled as simply supported (pinned – roll) while the lateral support was modeled by restraining the beam in its lateral direction.

The model was verified by comparing the critical moment for each model of the section between the value obtained from the formula for elastic buckling in Equation (5) and (6) and the value obtained from the analysis result of the linearised buckling analysis conducted by ADINA v9.3.2. For verification, lateral support was not modeled yet. Thus the beam section was analysed as a single span beam with a total length of L. Table 2 to Tabel 4 show the values of M_n derived from the formula in AISC and from the ADINA result for both steel sections with different load cases.

Table 2. Comparison of M_n value between AISC formula and ADINA analysis result for model A and B with uniformly distributed load.

Model No.	L (m)	M_n calculated using AISC formula (N.mm)	M_n obtained from ADINA model (N.mm)	% diff.
1A	8	96288035.58	101640000	5.56
2A	10	71738246.46	76160000	6.16
5A	11	63658821.63	66701250	4.78
3A	12	57244001.06	60530400	5.74
6A	13	52029325.05	54358850	4.48
4A	14	47706727.24	50969800	6.84
7A	15	44064523.34	46620000	5.80
8A	17	38262546.48	39954250	4.42
1B	8	181869054.94	185875200	2.20
2B	10	134016977.61	138460000	3.32
5B	11	118428612.01	123069100	3.92
3B	12	106124146.05	111384000	4.96
6B	13	96174865.31	99549450	3.51
4B	14	87967163.48	92472800	5.12
7B	15	81081148.38	85128750	4.99
8B	17	70171482.87	72979725	4.00

Table 3. Comparison of M_n value between AISC formula and ADINA analysis result for model A and B with concentrated load.

Model No.	L (m)	Mn calculated using AISC formula (N.mm)	Mn obtained from ADINA model	% diff.
1A	8	111491409.62	117468000	5.36
2A	10	83065338.01	88107500	6.07
5A	11	73710214.52	78650000	6.70
3A	12	66282527.54	70863000	6.91
6A	13	60244481.64	64811500	7.58
4A	14	55239368.38	59787000	8.23
7A	15	51022079.66	55380000	8.54
8A	17	44304001.19	44310500	0.01
1B	8	210585221.51	218660000	3.83
2B	10	155177553.03	169032500	8.93
5B	11	137127866.53	143715000	4.80
3B	12	122880590.16	128895000	4.89
6B	13	111360370.36	117286000	5.32
4B	14	101856715.61	107425500	5.47
7B	15	93883434.96	99401250	5.88
8B	17	81251190.69	86576750	6.55

Table 4. Comparison of Mn value between AISC formula and ADINA analysis result for model A and B with uniform end-moment.

Model No.	L (m)	Mn calculated using AISC formula (N.mm)	Mn obtained from ADINA model	% diff.
1A	8	84733471.31	87346740	3.08
2A	10	63129656.88	64165920	1.64
5A	11	56019763.04	58745310	4.87
3A	12	50374720.93	52928070	5.07
6A	13	45785806.05	48432930	5.78
4A	14	41981919.97	44202210	5.29
7A	15	38776780.54	40478295	4.39
8A	17	33671040.90	35344140	4.97
1B	8	160044768.35	161897580	1.16
2B	10	117934940.30	120089970	1.83
5B	11	104217178.57	106657200	2.34
3B	12	93389248.52	95739930	2.52
6B	13	84633881.47	87136920	2.96
4B	14	77411103.86	79841970	3.14
7B	15	71351410.57	73905390	3.58
8B	17	61750904.93	64396800	4.28

Fig 3 shows the buckling shape as a result from linearised buckling analysis of Model 1B with concentrated load at midspan.

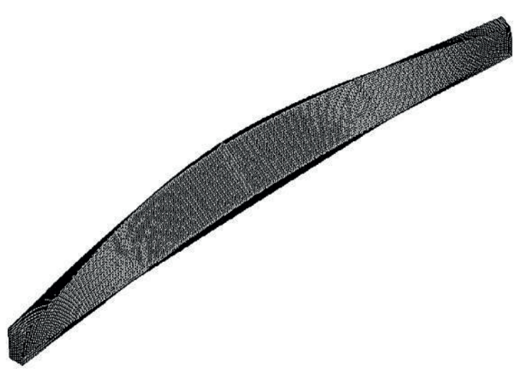

Fig. 3. Buckling shape from linearised buckling analysis of verification model (without lateral supports)

From these verification models, it was found that the difference between the results obtained by FEM models using ADINA and the results calculated using AISC formula were less than 10% and gave a reasonable buckling shape. Thus, the parameters and assumptions used to model the beam structure were deemed acceptable and could be used for further analysis.

3.2 Linearised buckling analysis for a finite element model of beam with lateral restraints

After carrying out verification, all the models were subsequently given the lateral supports with length of span as prescribed in Table 1. The same load cases were applied to all the models, and then the results for M_n from ADINA linearised buckling analysis were compared with the values calculated using the AISC formula [Equation (5)] for the elastic critical moment as shown in Tables 5 to 7 and Figures 5 to 7 for both steel sections with different load cases.

Figure 4 shows the buckling shape as a result from the linearised buckling analysis of beams with lateral restraints (Model 1B).

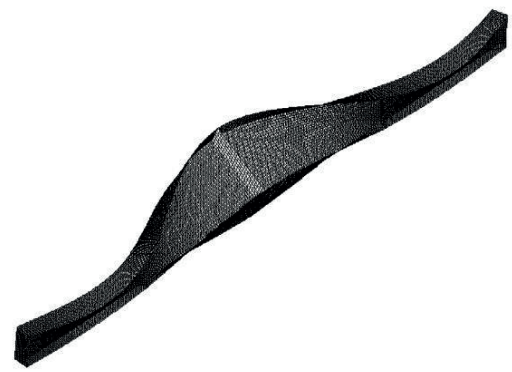

Fig. 4. Buckling shape from the linearised buckling analysis of model with lateral supports

Table 5. Comparison of M_n value between AISC formula for elastic critical moment and ADINA analysis result for model A and B with uniformly distributed load using linearised buckling analysis.

Model No.	L_b - L_s **(m)**	M_n **calculated using AISC formula (N.mm)**	M_n **obtained from ADINA model (N.mm)**	**% diff.**
1A	4 – 2	253520053.78	511616000	101.80
2A	4 – 3	250728438.18	408800000	63.04
3A	4 – 4	249237620.44	330724800	32.69
4A	4 – 5	248347241.47	315560000	27.06
5A	7 – 2	107373083.92	230807500	114.96
6A	7 – 3	105837684.38	206906700	95.49
7A	7 – 4	104896581.81	185692500	77.02
8A	7 – 5	104276904.53	166695200	59.86
1B	4 – 2	496410089.55	1004416000	102.34
2B	4 – 3	490943909.93	797300000	62.40
3B	4 – 4	488024783.98	638467200	30.83
4B	4 – 5	486281359.36	520399600	7.02
5B	7 – 2	204263677.40	449164100	119.89
6B	7 – 3	201342774.45	399617400	98.48
7B	7 – 4	199552446.15	357682500	79.24
8B	7 – 5	198373588.71	318268475	60.44

Table 6. Comparison of M_n value between AISC formula for elastic critical moment and ADINA analysis result for model A and B with concentrated load using linearised buckling analysis.

Model No.	L_b - L_s (m)	M_n calculated using AISC formula (N.mm)	M_n obtained from ADINA model (N.mm)	% diff.
1A	4 – 2	279448241.10	553280000	97.99
2A	4 – 3	272029261.25	462150000	69.89
3A	4 – 4	267298317.57	390000000	45.90
4A	4 – 5	264018583.62	368550000	39.59
5A	7 – 2	120569541.76	248784250	106.34
6A	7 – 3	117316139.41	224727750	91.56
7A	7 – 4	115039734.76	204847500	78.07
8A	7 – 5	113357691.64	187601375	65.50
1B	4 – 2	547179303.25	1080560000	97.48
2B	4 – 3	532652419.10	901225000	69.20
3B	4 – 4	523388898.76	757380000	44.71
4B	4 – 5	516966949.09	642460000	24.27
5B	7 – 2	229368265.14	482625000	110.41
6B	7 – 3	223179079.70	434330000	94.61
7B	7 – 4	218848508.50	394241250	80.14
8B	7 – 5	215648634.75	360064250	66.97

Table 7. Comparison of M_n value between AISC formula for elastic critical moment and ADINA analysis result for model A and B with uniform end-moment using linearised buckling analysis.

Model No.	L_b - L_s (m)	M_n calculated using AISC formula (N.mm)	M_n obtained from ADINA model (N.mm)	% diff.
1A	4 – 2	245914452.17	626675400	154.83
2A	4 – 3	245914452.17	453039600	84.23
3A	4 – 4	245914452.17	385876920	56.92
4A	4 – 5	170141736.07	258470550	51.91
5A	7 – 2	102155284.47	221760240	117.08
6A	7 – 3	102155284.47	189545070	85.55
7A	7 – 4	102155284.47	160987710	57.59
8A	7 – 5	102155284.47	137278050	34.38
1B	4 – 2	481517786.86	890487000	84.93
2B	4 – 3	481517786.86	640949400	33.11
3B	4 – 4	481517786.86	596475360	23.87
4B	4 – 5	329602317.11	363288510	10.22
5B	7 – 2	194337475.55	431257320	121.91
6B	7 – 3	194337475.55	366005250	88.33
7B	7 – 4	194337475.55	309658050	59.34
8B	7 – 5	194337475.55	262618200	35.14

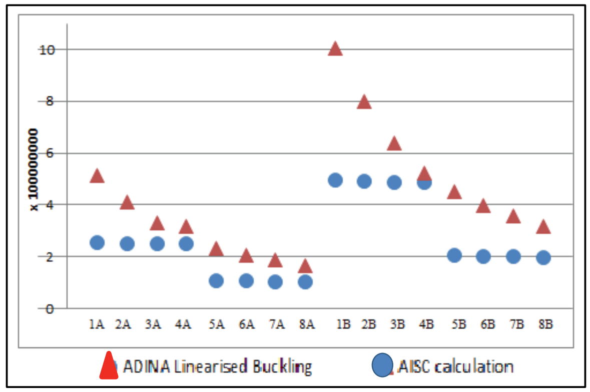

Fig. 5. Comparison of AISC calculation and linearised buckling analysis results for ADINA Model A and B with uniformly distributed load

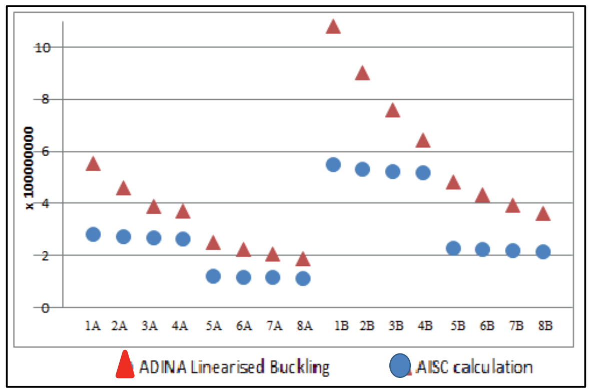

Fig. 6. Comparison of AISC calculation and linearised buckling analysis results for ADINA Model A and B with concentrated load

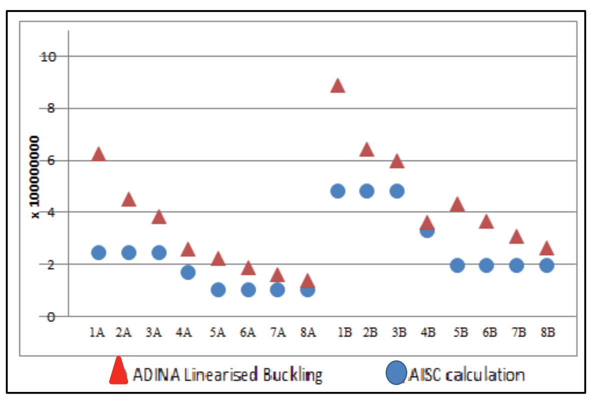

Fig. 7. Comparison of AISC calculation and linearised buckling analysis results for ADINA Model A and B with uniform end-moment

3.3 Collapse analysis for finite element model of beam with lateral restraints

As stated in Section 3.1, the collapse analysis was also carried out to reflect the real condition better, where the beam is in fact not perfectly elastic (possesing yield stress), possessing geometrical imperfection (not perfectly straight) and also the load is not always applied at the shear centre. However, in this study, only geometrical imperfection was being considered, while the load was applied centrically. Tables 8 to 10 and Figures 9 to 11 also show the comparison of Mn from the ADINA collapse analysis and calculation using the AISC formula based on respective L_b [Equation (4) and (5)].

Figure 8 shows the buckling shape as a result from the collapse analysis of beams with lateral restraints (Model 1B).

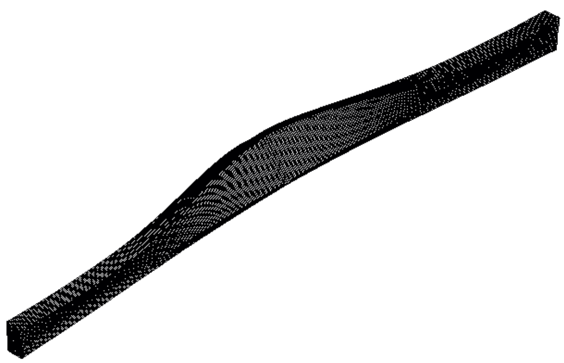

Fig. 8. Buckling shape from the collapse analysis of model with lateral supports

Table 8. Comparison of M_n value between AISC formula and ADINA analysis result for model A and B with uniformly distributed load using collapse analysis.

Model No.	L_b - L_s (m)	M_n calculated using AISC formula (N.mm)	M_n obtained from ADINA model (N.mm)	% diff.
1A	4 – 2	175756796.50	183680000	4.51
2A	4 – 3	173821464.73	180600000	3.90
3A	4 – 4	172787931.70	176400000	2.09
4A	4 – 5	172170662.36	175616000	2.00
5A	7 – 2	107373083.92	150766000	40.41
6A	7 – 3	105837684.38	142669800	34.80
7A	7 – 4	104896581.81	135450000	29.13
8A	7 – 5	104276904.53	129472000	24.16
1B	4 – 2	285570770.35	310643200	8.78
2B	4 – 3	282426230.87	296520000	4.99
3B	4 – 4	280746939.77	289296000	3.05
4B	4 – 5	279743996.59	286748000	2.50
5B	7 – 2	204263677.40	254184700	24.44
6B	7 – 3	201342774.45	242515000	20.45
7B	7 – 4	199552446.15	231840000	16.18
8B	7 – 5	198373588.71	221923100	11.87

Table 9. Comparison of M_n value between AISC formula and ADINA analysis result for model A and B with concentrated load using collapse analysis.

Model No.	L_b - L_s (m)	M_n calculated using AISC formula (N.mm)	M_n obtained from ADINA model (N.mm)	% diff.
1A	4 – 2	193731923.42	213590000	10.25
2A	4 – 3	188588598.02	195942500	3.90
3A	4 – 4	185308796.31	191685000	3.44
4A	4 – 5	183035068.75	187141500	2.24
5A	7 – 2	120569541.76	161447000	33.90
6A	7 – 3	117316139.41	153367500	30.73
7A	7 – 4	115039734.76	145762500	26.71
8A	7 – 5	113357691.64	140915125	24.31
1B	4 – 2	314776871.86	334646000	6.31
2B	4 – 3	306419963.76	328607500	7.24
3B	4 – 4	301090920.91	319605000	6.15
4B	4 – 5	297396553.78	317908500	6.90
5B	7 – 2	229368265.14	278671250	21.50
6B	7 – 3	223179079.70	262837250	17.77
7B	7 – 4	218848508.50	251891250	15.10
8B	7 – 5	215648634.75	241995000	12.22

Table 10. Comparison of M_n value between AISC formula and ADINA analysis result for model A and B with uniform end-moment using collapse analysis.

Model No.	L_b - L_s (m)	M_n calculated using AISC formula (N.mm)	M_n obtained from ADINA model (N.mm)	% diff.
1A	4 – 2	170484092.61	191572290	12.37
2A	4 – 3	170484092.61	182890500	7.28
3A	4 – 4	170484092.61	180114090	5.65
4A	4 – 5	150949206.61	171873000	13.86
5A	7 – 2	102155284.47	136528860	33.65
6A	7 – 3	102155284.47	130359060	27.61
7A	7 – 4	102155284.47	122690880	20.10
8A	7 – 5	102155284.47	113348040	10.96
1B	4 – 2	277003647.24	301910310	8.99
2B	4 – 3	277003647.24	299797290	8.23
3B	4 – 4	277003647.24	295923420	6.83
4B	4 – 5	251437539.21	287119170	14.19
5B	7 – 2	194337475.55	225992520	16.29
6B	7 – 3	194337475.55	216182070	11.24
7B	7 – 4	194337475.55	205566660	5.78
8B	7 – 5	194337475.55	193089780	-0.64

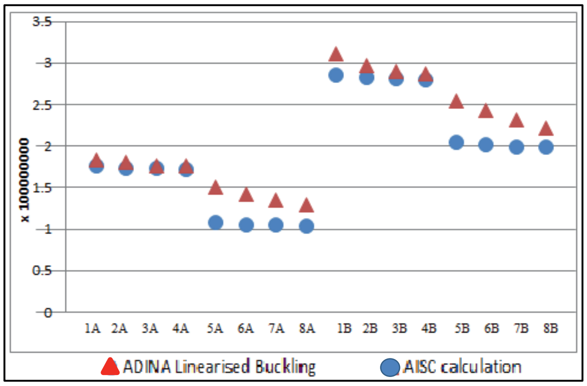

Fig. 9. Comparison of AISC calculation and collapse analysis results for ADINA Model A and B with uniformly distributed load

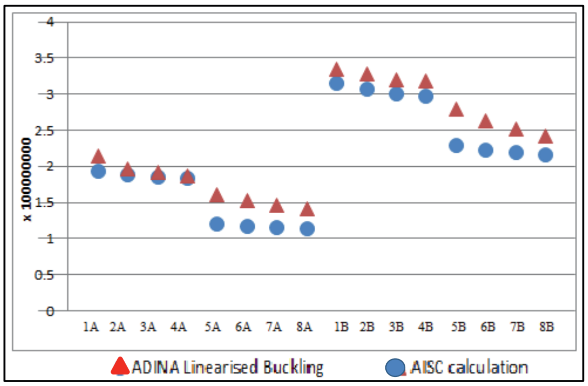

Fig. 10. Comparison of AISC calculation and collapse analysis results for ADINA Model A and B with concentrated load

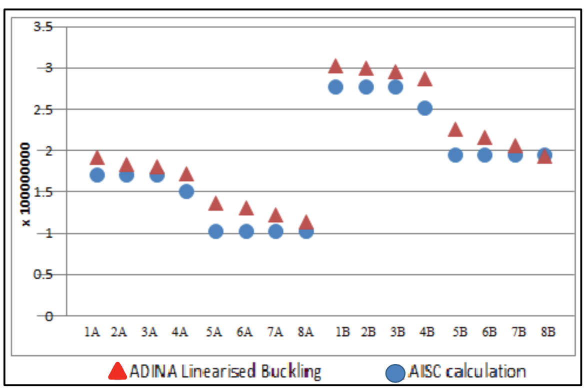

Fig. 11. Comparison of AISC calculation and collapse analysis results for ADINA Model A and B with uniform end-moment

4 Discussion and Conclusion

Based on the results obtained from the finite element analysis using ADINA, it was found that there is an effect from adjacent segments of steel beam with lateral supports on the flexural strength of a beam spanning in between lateral supports. Generally, the flexural strength was in fact increased by the presence of adjacent segments of the beam.

In comparison with the linearised buckling analysis results, the increase ranged from a minimum increase of 7.02% to a maximum increase of 154.83% for all load cases, while compared to the collapse analysis results, the difference ranged from a minimum increase of 2.00% to a maximum increase of 40.41% for all load cases.

The results from the linearized buckling analysis showed quite a significant increase in the flexural strength of the beam spanning in between lateral supports due to elastic stiffness of the boundary of the considered segment contributed by the adjacent segments, as shown in Figure 11 to Figure 14. However, as the adjacent segments became longer, the elastic stiffness decreased and thus the increase of its critical moment was less. As stated in the discussion above, the results of the linearised buckling analysis were using assumptions which do not reflect the actual condition of the steel I-beam, i.e. being not always perfectly elastic.

In the collapse analysis, the increase was not as high compared to the linearised buckling results, as shown in Figures 15 to Figure 18. The result from its collapse analysis is in fact a better approach to the numerical analysis assumptions. The steel I-beam material is not always perfectly elastic, it is not geometrically perfect, and the load is not perfectly applied at the centroid, although in this study the load is applied centrically. The span of beam L_b also plays a role in determining whether the beam structure is in the elastic or inelastic region. Despite these, the trend remained. As the adjacent segments became longer, the increase was less due to the yielding experienced first by the adjacent segments, which means less contribution to the boundary condition of the considered beam segment.

Thus, it may be concluded that the value of flexural strength of a beam spanning in between lateral supports was actually higher than the value expected by calculating this using the AISC formula due to the effect of the adjacent beam segments. The collapse analysis performed in this study showed an increase of 2.00% to 40.41% for all load cases which are a uniformly distributed load, a concentrated load, and a uniform end-moment. As the adjacent segment became longer, the stiffness decreased and the contribution to the restraint of the beam in between lateral supports was less, indicating less increase of the critical moment.

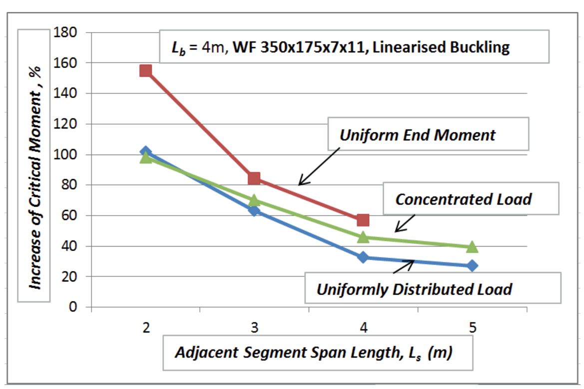

Fig. 11. L_s against the % increase of critical moment for L_b = 4m (Model A) in linearised buckling analysis for all load cases.

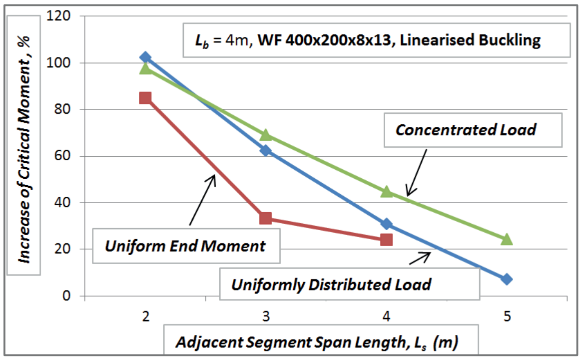

Fig. 12. L_s against the % increase of critical moment for L_b = 4m (Model B) in linearised buckling analysis for all load cases.

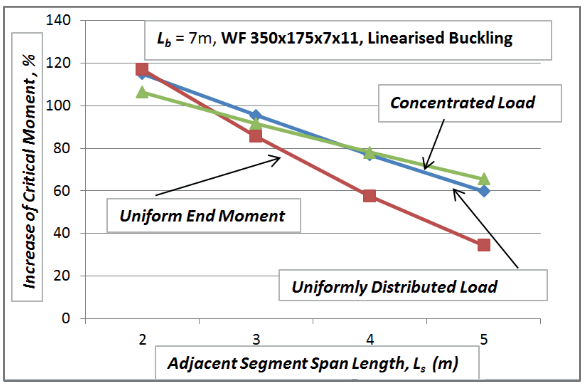

Fig. 13. L_s against the % increase of critical moment for L_b = 7m (Model A) in linearised buckling analysis for all load cases.

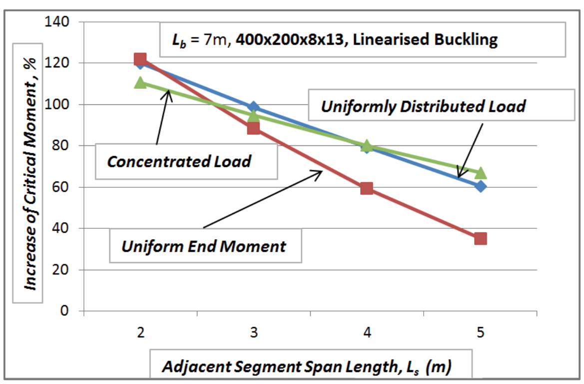

Fig. 14. L_s against the % increase of critical moment for L_b = 7m (Model B) in linearised buckling analysis for all load cases.

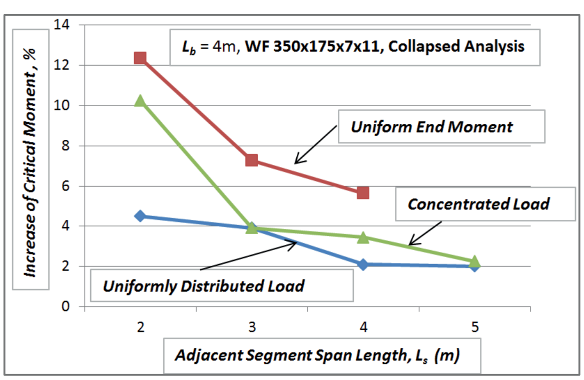

Fig. 15. L_s against the % increase of critical moment for L_b = 4m (Model A) in collapse analysis for all load cases.

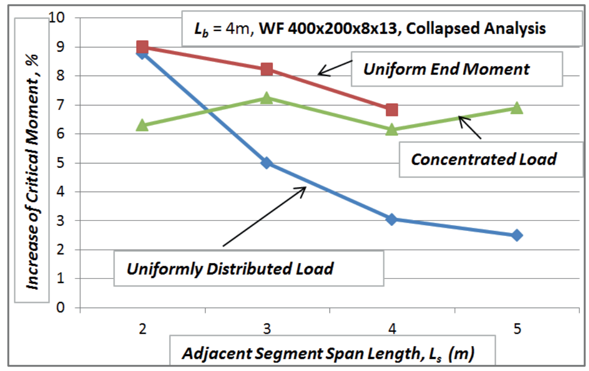

Fig. 16. L_s against the % increase of critical moment for L_b = 4m (Model B) in collapse analysis for all load cases.

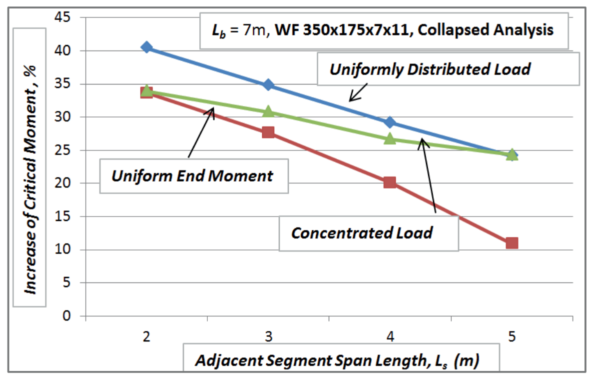

Fig. 17. L_s against the % increase of critical moment for L_b = 7m (Model A) in collapse analysis for all load cases.

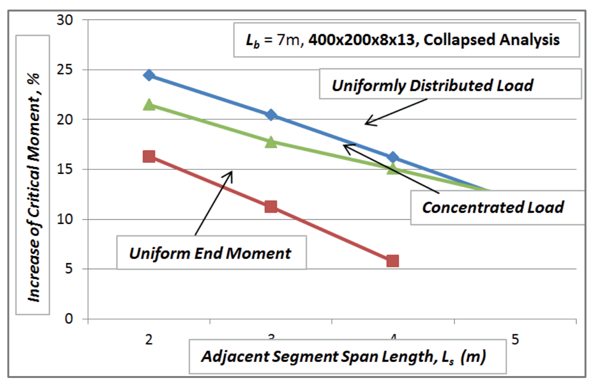

Fig. 18. L_s against the % increase of critical moment for L_b = 7m (Model B) in collapse analysis for all load cases.

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