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Modeling considerations for built-up compression members in communication towers

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Abstract
It is a well-established fact that built-up compression members exhibit extra flexibility than their equivalent solid members due to the shear action which effects the overall stability. This is usually taken into consideration by using a modified slenderness ratio for buckling modes that involve relative deformation of the interconnectors. Built-up members are often used as space truss members in communication towers. When modeling communication towers in structural analysis programs like SAP2000, the built-up member is usually defined as a single member element with a double section cross section. This does not catch the increase in deflection in the member due to the shear effect on the interconnected main members. To investigate the effect of this extra flexibility of built-up members on the overall side-sway of a communication tower, the single member element can be replaced by two elements with a variable number of interconnectors. The nonlinear analysis of the tower with a single element representing the built-up member was compared with the nonlinear analysis of the tower using two elements to represent the built-up member. Communication towers with a height of 22 m, 50 m, and 100 m were investigated to see the accuracy of the results obtained by modeling built-up compression members as a single member. Recommendations are made for the correction of the side-sway of communication towers modeled using single elements to represent built-up members.

1. Introduction

Built-up compression members, composed of two or more rolled sections interconnected by batten plates, are commonly used in space trusses and bracing members. From previous research (Elmahdy 2008a) these members can be analyzed either using a $P\delta$ analysis or more accurately using a shear analysis, depending on the orientation of the axial load. In both analyses, it can be seen, from Fig. 1, that the built-up member exhibits an increased flexibility which increases its deflection when buckling occurs about an axis passing through the open web. This increased flexibility is due to the shear deformation between interconnectors and is accounted for by using a modified or equivalent slenderness ratio (Timoshenko and Gere 1961). The general format for this modified slenderness ratio, Λ_m, is to take the square root of the square of the integral slenderness ratio about the open web, Λ_o, plus the square of the slenderness ratio of the

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individual member between interconnectors, Λ_i, multiplied by a factor K_i, as given in Eq. 1. The exact value of the K_i factor depends on many factors including the type of interconnector and the type of its connection to the main members and is currently under investigation (Elmahdy 2008b and El-Mahdy 2019).

$$\Lambda_m = \sqrt{\Lambda_o^2 + \left(K_i \Lambda_i\right)^2} \tag{1}$$

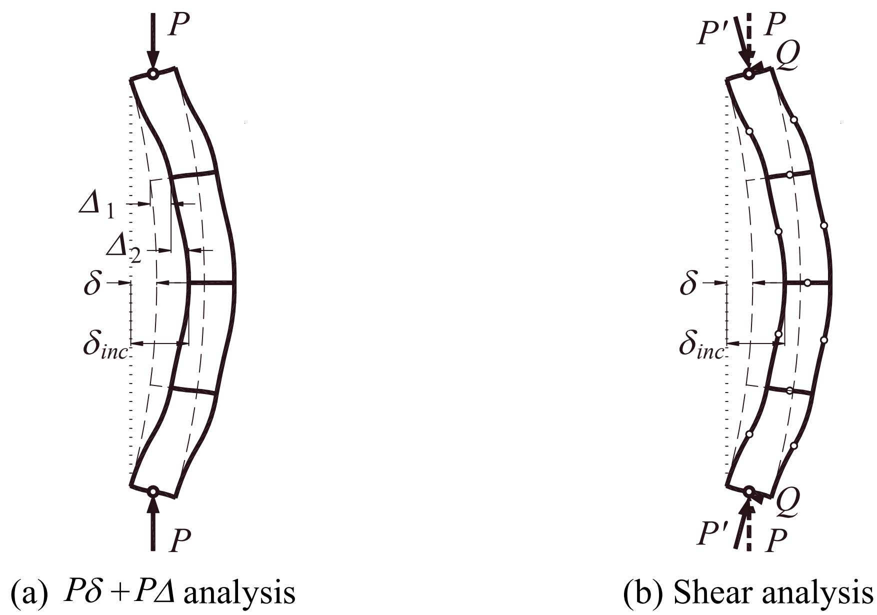


Figure 1: Increased deflection in built-up compression members

The increase in flexibility of a built-up compression member is easily obtained when the member is modeled as a member consisting of many elements, individual members and interconnectors. However, for more complex structures such as communication towers, as shown in Fig. 2, which consist of many built-up members, it is common practice to model these members as integral members having built-up cross sections that are available in structural analysis programs such as SAP2000. This leads to results in the side-sway of communication towers that are less than the real values for these structures under wind loads, as the compound behavior of the built-up member is not taken into account.

Previous research has been conducted on communication towers (Carril *et al*. 2003, Zhuge *et al*. 2012, Szafran and Rykaluk 2016, and Martín *et al*. 2016), however the increase in side-sway due to the compound nature of built-up members has not been investigated before. Carril *et al.* (2003) conducted an experimental study on the wind forces on rectangular lattice communication towers with antennas. Zhuge *et al.* (2012) studied the modeling of steel lattice tower angle legs reinforced for increased load capacity. Szafran and Rykaluk (2016) conducted a full-scale experiment on a lattice communication tower under breaking load. Finally, Martín *et al.* (2016) conducted an experimental study of the effects of dish antennas on the wind loading of telecommunication towers.

In this paper communication towers with three heights, 22 m, 50 m, and 100 m, are modeled in the SAP2000 software, once using single elements for the space truss members with a cross section of double angles, then again using double elements for the built-up space truss members with a varying number of interconnectors. The results of the side-sway for the double element towers are compared to those for the single element tower and a correction factor for the side-sway is suggested.

(a) Short communication tower

(b) Tall communication tower

Figure 2: Examples of communication towers

2. Validation of the SAP2000 program results

To validate the SAP2000 software results a simple cantilever post of length 10 m was modeled, first as a single member with a double angle cross section of 2L 100x100x10 and spacing between the angles of 10 mm, then as two single members of single angle cross section L 100x100x10 separated by a distance of 2*e* + 10 mm, *e* being the distance of the centroid from the outer leg of the angle. The number of interconnectors was varied for this second model from zero to five. For each model the element size of the vertical elements was kept constant at approximately 10 elements along the length regardless of the number of interconnectors. This was to eliminate the effect of mesh refinement with the increase of the number of interconnectors. A horizontal load was applied to the end of the cantilever post of 10 kN and the own weight (O.W.) of the post was applied as an axial load. A nonlinear analysis was conducted for each model under the effect of the horizontal load and its own weight. Fig. 3 shows the statical system and cross section of the validation model and Fig. 4 shows the validation models.

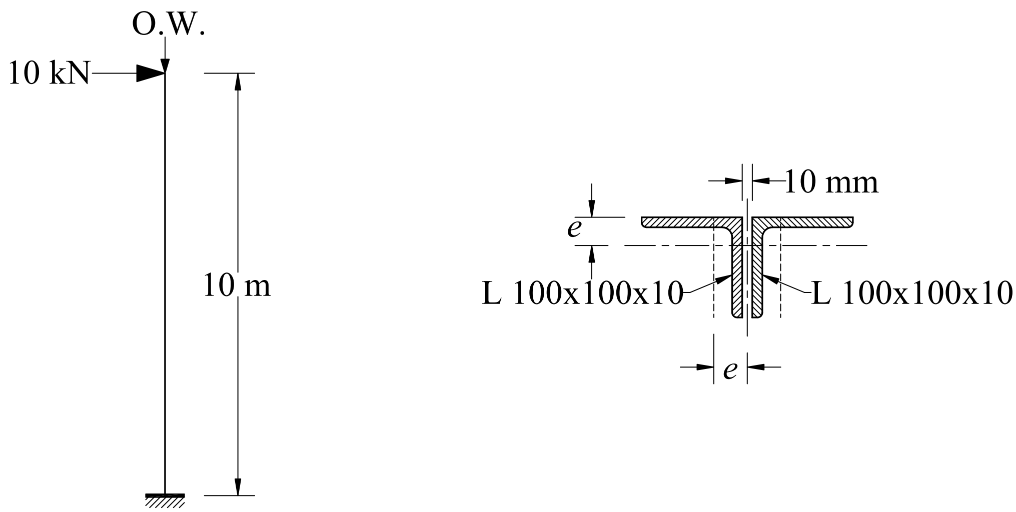
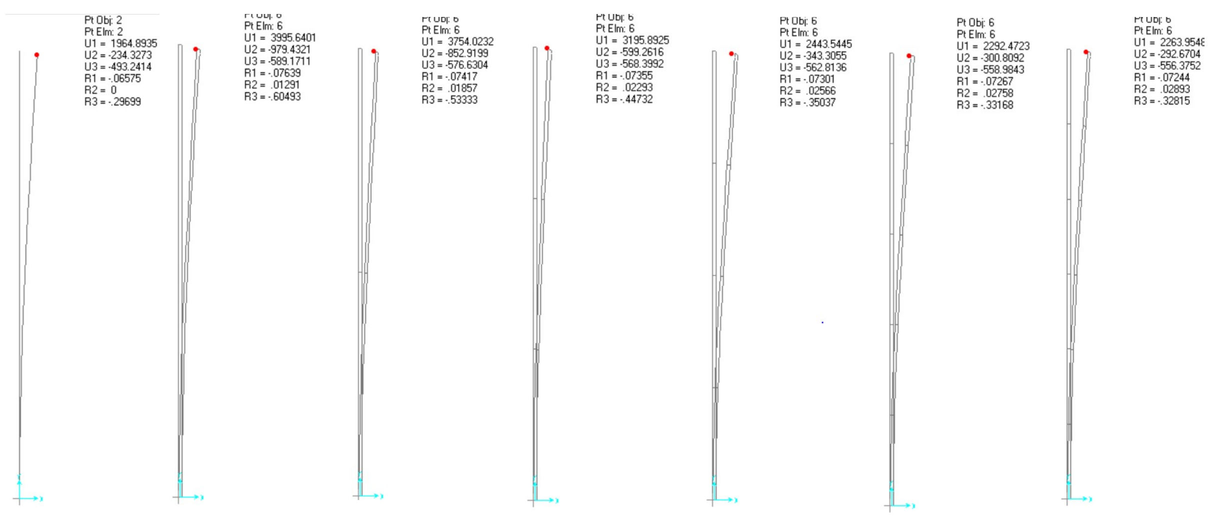


Figure 3: Statical system and cross section of validation model



Double angle | Zero interconnectors | One interconnector | Two interconnectors | Three interconnectors | Four interconnectors | Five interconnectors

Figure 4: SAP2000 validation models of a cantilever post with varying numbers of interconnectors

The side-sway of each case was determined under the effect of the horizontal load and compared to the theoretical deflection of $PL^3/3EI$, where P is the 10 kN horizontal load, L is the length of the cantilever which is 10 m, E is the modulus of elasticity taken as 210 GPa, and I is the moment of inertia of the cross section taken as 8 x 10^6 mm^4. This gave a theoretical side-sway of 1984 mm. The effect of the trivial own weight was neglected in the side-sway calculation as this was much less than the magnitude of the horizontal load. Table 1 gives the side-sway of each model as compared to the theoretical value of the deflection. It can be seen from Table 1 that the SAP2000 program accurately predicts the side-sway for the model with double angles, however the models with single angles and interconnectors show an increase in side-sway compared to the theoretical value of the side-sway. This increase in side-sway decreases with the increase in number of interconnectors to approximately approach the value of side-sway for the double angle member, as can be seen from Fig. 5.

Table 1: Values of side-sway for each validation model compared to theoretical side-sway

Model	FE side-sway (mm)	FE side-sway / Theoretical side-sway
Double angle	1965	0.99
Zero interconnectors	3997	2.02
One interconnector	3754	1.89
Two interconnectors	3196	1.61
Three interconnectors	2444	1.23
Four interconnectors	2293	1.16
Five interconnectors	2264	1.14

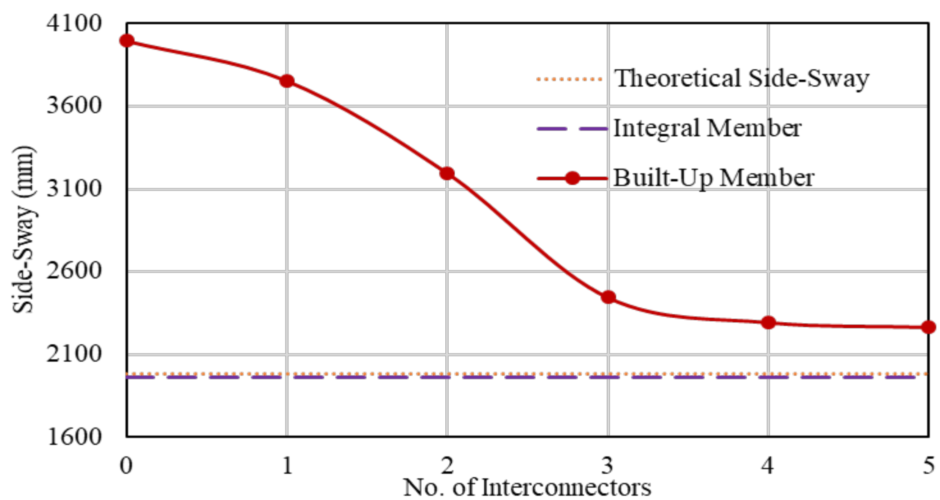
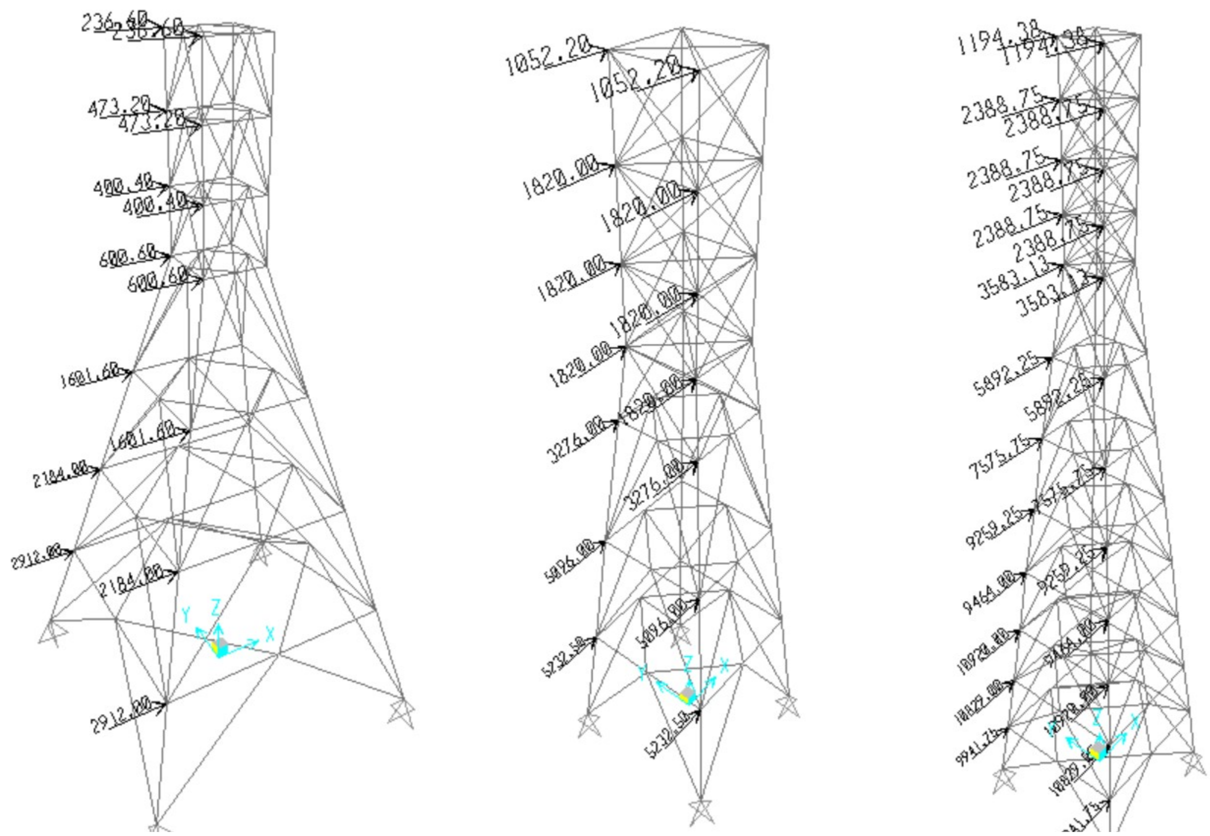


Figure 5: Plot of sidesway for validation models

3. Finite element modeling of the towers using SAP2000

Three towers were modeled using SAP2000 having a height of 22 m, 50 m, and 100 m. For each tower model, the vertical members were modeled six times, one for double angle members 2L 100x100x10, and five times for two single angle members spaced at the same distance as the double angle member. For these single angle members, the number of interconnectors was varied from zero to five interconnectors. The mesh size of the vertical members was taken the same as the validation model, approximately 10 elements for each member. Fig. 6 shows the models of the three towers, which are not drawn to scale.

The 22 m high tower had a square base of 10 m and a square peak level of 2 m and consisted of seven levels as given in Table 2. The 50 m high tower had a square base of 12 m and a square peak level of 5 m and consisted of seven levels as given in Table 3. The 100 m high tower had a square base of 21 m and a square peak of 5 m and consisted of 12 levels as given in Table 4. The horizontal and vertical diagonal bracing were taken as 2L 60x60x6 members and were modelled as single element members.



(a) 22 m high tower (b) 50 m high tower (c) 100 m high tower

Figure 6: Models of the three towers with wind loads (not to scale)

For each tower the wind loads were calculated according to the Egyptian Code of Practice for the Calculation of Forces and Loads on Structural Works and Buildings (ECP 2012). Table 2 shows the dimensions and wind loads applied to the 22 m tower at each level. Table 3 shows the dimensions and wind loads applied to the 50 m high tower at each level. Table 4 shows the dimensions and wind loads applied to the 100 m high tower at each level.

Table 2: Wind loads applied to the 22 m high tower at each level according to ECP (2012)

Level	**Height (m)**	**Width (m)**	**Wind load (kN)**
0	0	10	-
1	4	8	2912
2	4	6	2184
3	4	4	1602
4	4	2	601
5	2	2	400
6	2	2	473
7	-	2	237

Table 3: Wind loads applied to 50 m high tower at each level according to ECP (2012)

Level	Height (m)	Width (m)	Wind load (kN)
0	0	12	-
1	10	10	5233
2	10	8	5096
3	10	6	3276
4	5	5	1820
5	5	5	1820
6	5	5	1820
7	-	5	1052

Table 4. Wind loads applied to 100 m high tower at each level according to ECP (2012)

Level	Height (m)	Width (m)	Wind load (kN)
0	10	21	-
1	10	19	9942
2	10	17	10829
3	10	15	10920
4	10	13	9464
5	10	11	9259
6	10	9	7575
7	10	7	5892
8	5	5	3583
9	5	5	2389
10	5	5	2389
11	5	5	2389
12	-	5	1194

Each model was run in SAP2000 under the effect of its own weight and the wind loads using a nonlinear analysis to apply the wind loads in order to include the effect of nonlinear geometry on the side-sway. The peak side-sway deformation and base shear was recorded for each model.

4. Results and discussion

All the models reached their final wind loads which was checked by comparing the base shear with the wind loads. So premature failure due to the local buckling of one of the chord members was not the mode of failure. This allowed for the comparison of the peak side-sway of the model with one double angle member to the peak side-sways of the models with two single angle members and with varying number of interconnectors. Table 5 gives the peak side-sway of each model, which is also plotted in Figs. 7-9.

From Table 5 it can be seen that using a single double angle member to model the tower's vertical members does not reflect the actual peak side-sway that would occur in the tower if the built-up member were properly modeled as single angle members with interconnectors. This means that a correction factor is required for the deflections of tower structures that are modeled as integral members as is the usual case in engineering practice. From the values in Table 5 and the plots shown in Fig. 7 this correction factor for the 22 m high tower is 1.57. Also, from the values given in Table 5 and the plots shown in Fig. 8 for the 50 m high tower the correction factor is 1.08. Finally, from the values given in Table 5 and the plots shown in Fig. 9 for the 100 m high tower the correction factor is 1.06. The results show that the side-sways are logically smaller in shorter towers being in the range of 6.5 mm for the 22 m high tower, 30.5 mm for the 50 m high tower, and 203 mm for the 100 m high tower. However, the error in peak side-sway for towers modeled using double angle member elements is more significant in shorter towers than in very tall towers.

In practice, the design and analysis of a communication tower would be done by modeling the tower as a space truss using single members with compound sections for each of the truss members. Although this would adequately give the forces in the members and hence be sufficient for the design of the tower, it would underestimate the amount of deformation at the peak of the tower under wind loads, leading to serviceability considerations. As most communication towers in industry are in the range of 20 – 30 m, this means there is a significant correction factor of approximately 1.5 that must be applied to the results of the standard finite element analysis.

Table 5: Results of peak side-sway for each model

Model	22 m model peak side-sway (mm)	50 m model peak side-sway (mm)	100 m model peak side-sway (mm)
Double angle members	4.18	28.33	192.26
Single angle members – zero interconnectors	6.59	30.81	204.83
Single angle members – one interconnector	6.56	30.60	203.42
Single angle member – two interconnectors	6.56	30.50	203.01
Single angle member – three interconnectors	6.56	30.45	202.78
Single angle members – four interconnectors	6.56	30.42	202.64
Single angle members – five interconnectors	6.56	30.40	202.55

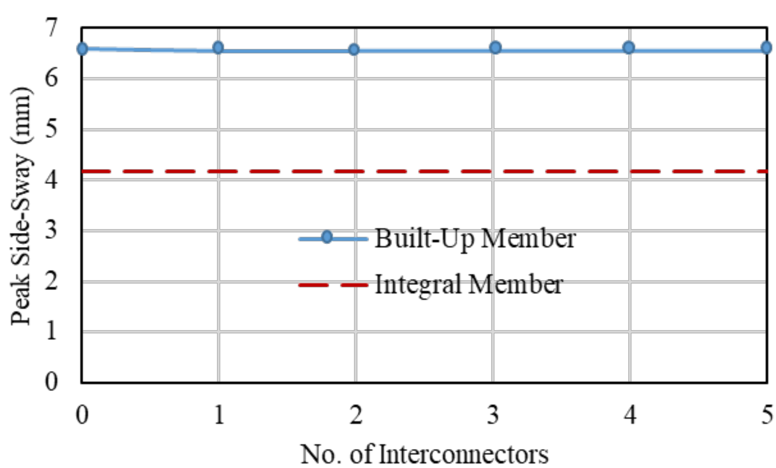


Figure 7: Comparison of peak side-sways of built-up members with integral member for 22 m high tower

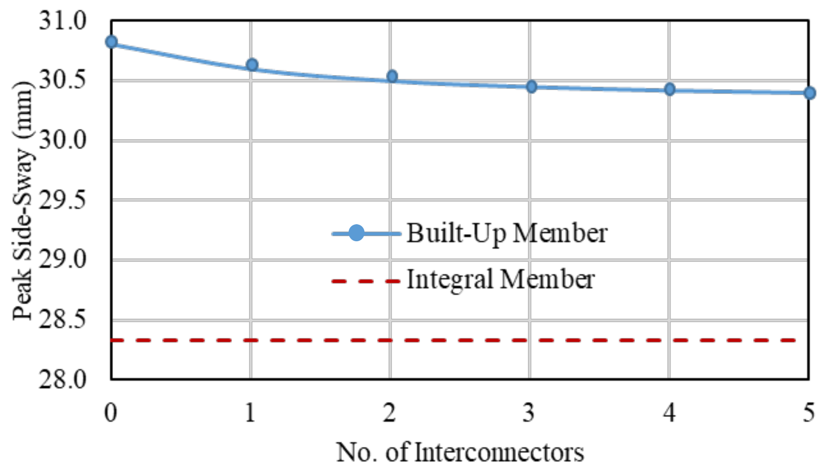


Figure 8: Comparison of peak side-sways of built-up members with integral member for 50 m high tower

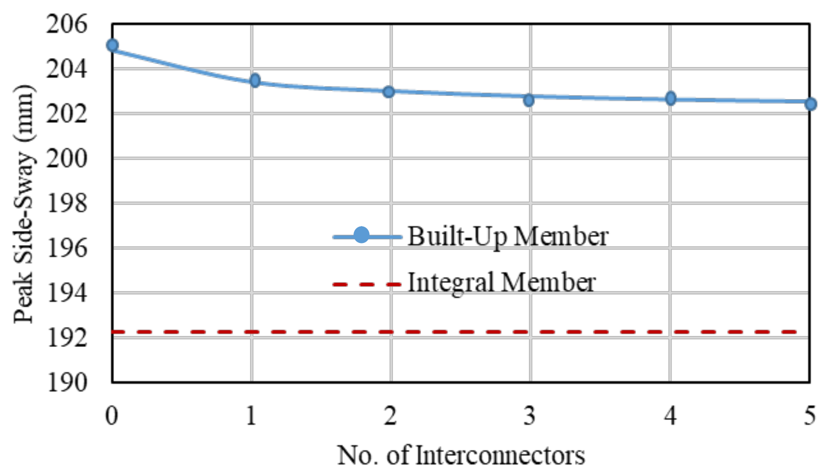


Figure 9: Comparison of peak side-sways of built-up members with integral member for 100 m high tower

5. Conclusions and recommendations

It is widely accepted that built-up members consisting of two steel sections interconnected together along the open web exhibit an increase in flexibility due to the effect of shear on the open web. This type of member is commonly used in space trusses such as communication towers. However, it is common practice in civil engineering to model these members as single members with double section cross sections due to the complexity of the structure. In this paper, an analysis was conducted to investigate this increased flexibility on the peak side-sway of communication towers by modeling the built-up member as double members with interconnectors. It was found that naturally the peak side-sway for taller towers was much greater than for shorter towers. However, the error in peak side-sway was more significant for shorter towers than taller towers. From the results of the analysis, it is recommended that a correction factor of 1.5 be used to get the actual deflection for towers up to 35 m in height and a correction factor of 1.08 be used for taller towers.

References

Carril, C.F., Isyumov, N., and Brasil, R.M.L.R.F. (2003). "Experimental study of the wind forces on rectangular latticed communication towers with antennas." *Journal of Wind Engineering and Industrial Aerodynamics*, Elsevier, 91: 1007-1022.

ECP. (2012), "Egyptian code of practice for the calculation of forces and loads on structural works and buildings." (*ECP 201*), Housing and Building National Research Center, Giza, Egypt.

Elmahdy, G. (2008a). "Contemporary theories of built-up compression members." *Proceedings of 2008 SSRC Annual Stability Conference*, Nashville, TN, USA. 19-38.

Elmahdy, G. (2008b). "Innovative method for calibrating built-up compression members." *Proceedings of the 2008 CSCE 37th Annual General Conference*, Quebec City, Quebec, Canada. 1028-1038.

El-Mahdy, G.M. (2019). "Experimental calibration of centrally loaded built-up battened columns." *Proceedings of the International Colloquium on the Stability and Ductility of Steel Structure (SDSS 2019)*, Wald & Jandera (Eds.), Prague, Czech Republic, ISBN 978-0-367-33503-8, pp 394-401.

Martín, P., Elena, V.B., Loredo-Souza, A.M., and Camaňo, E.B. (2016). Experimental study of the effects of dish antennas on the wind loading of telecommunication towers." *Journal of Wind Engineering and Industrial Aerodynamics*, Elsevier, 149: 40-47.

Szafran, J. and Rykaluk, K. (2016). "A full-scale experiment of a lattice telecommunication tower under breaking load." *Journal of Constructional Steel Research*, Elsevier, 120:160-175.

Timoshenko, S.P. and Gere, J.M. (1961). "Theory of elastic stability." *McGraw Hill Book Company, Inc*, New York.

Zhuge, Y., Mills, J. E., and Ma, X. (2012). "Modelling of steel lattice tower angle legs reinforced for increased load capacity." *Engineering Structures*, Elsevier, 43: 160-168.