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APPLICATION OF WIND MOMENT METHOD ON THE DESIGN OF MULTI-STOREY UNBRACED STEEL FRAMES WITH PARTIAL AND FULL STRENGTH CONNECTIONS

M. M. Tahir, P. N. Shek, C. S. Tan, A. Saggaff
Steel Technology Centre, Universiti Teknologi Malaysia, 81310 Skudai, Johor
E-mail: mahmoodmdtahir@yahoo.com

ABSTRACT: In the multi-storey unbraced steel frame, the requirements of lateral load resistance are very critical in the design. Shear wall, core wall and bracing system are commonly applied to resist the lateral loads. However in some cases, due to the architectural requirements, the frames have to be designed as unbraced. As a result, the lateral load due to wind needs to be catered by utilizing the stiffness of connections, columns and beams. The application of wind moment method on the design of unbraced frame has becoming popular due its simplicity and straight forward approach. The objective of this paper is to compare the design of unbraced frames between partial and full strength connections with column bending on major axis. The frames were designed to satisfy the ultimate limit state and service limit state based on BS5950-1:2000. The sway-deflection of the frame was limited to $h_T/450$ for partial strength connections and $h_T/300$ for full strength connections, where h_T is the total height of the multi-storey frame. The economic aspect was presented base on the total steel weight savings of the unbraced plane frame design using both partial and full strength connections. A parametric study on a series of two bays with two, four, six, and eight storey height were done. It was concluded that the total steel weight saving for frame design with wind moment method using full strength connection was up to 21% less than the frame design using partial strength connections.

Keywords —Unbraced frame, wind moment method, partial strength connections, full strength connections.

1. INTRODUCTION

Multi-storey frames may be divided into two distinct categories for the purpose of design: sway and non-sway frames. In BS 5950-1: 2000, a multi-storey frame may be classified as "non-sway" if its sway deformation is small for the resulting secondary forces and moments to be negligible. In Eurocode 3, the frame is classified as braced when the bracing system reduces the horizontal displacement by at least 80%. A steel frame which does not satisfy the criterion for a braced frame is classified as unbraced. For an unbraced frame, the main consideration is to limit sway, to control the inter-storey drifts and to avoid premature collapse by frame instability. To meet this requirement, it is usual to rely on the bending resistance and stiffness of the connections to resist horizontal loads. For ultimate limit state, it is important to make sure that the structural members are capable of transferring the factored loads to the columns and down to the foundations. In practice, unbraced frame usually designed by assuming that the connections are rigid in order to provide adequate stiffness to resist horizontal loads. In rigid frame analysis and design, the internal moments and forces are distributed among the columns and beams according to their stiffness coefficients (K). The stiffness coefficient is a function of the length (L), the second moment of area (I) and the modulus of elasticity (E).

One alternative, a simple design method, termed the wind moment method is often used in the U.K. for the design of unbraced frame. Wind moment method, also known as wind connection method, assuming that the structure is statically determinate and allow the structure to be analyzed using manual techniques. The designed method proposed in wind moment method assuming that the connections act as pins under gravity load and rigid under horizontal loads. These assumptions allow the beams and columns to be designed using simple construction methods and sway deflections are calculated using the simple graphical method assuming connections is rigid. As the beam in wind moment design usually governed though by mid-span gravity moment, the connections are designed to a lower moment than the beam sections and are therefore termed as partial strength in the context of Eurocode 3 Part 1.1. The partial strength connections proportioned in wind moment design have some degree of strength and stiffness, but insufficient to develop full continuity as in rigid connection. The standard tables for these types of connections have been produced by the Steel Construction Institute (Joints in steel construction: Moment connections). Both rigid and partial strength joints can be applied in wind moment design where the controlling parameter is the sway limit at serviceability limit state. The calculated rigid frame deflections will be increased by 50% as an approximate allowance for partial strength connections as suggested by SCI. The main advantage of the wind moment method is its simplicity. The frame is treated as statically determinate, thus the internal moments and forces are not dependent on the relative stiffness of the frame members. The need of iterative analysis and design procedure is therefore avoided.

This paper intended to compare the design of unbraced frames bending on major axis between partial and full strength connections. The frames were designed to satisfy both the ultimate limit state and serviceability limit state based on BS 5950-1: 2000, but limit by the sway-deflection of h_T / 450 for partial strength connections and h_T / 300 for full strength connections, where h_T is the total height of the multi-storey frame. The objective of this paper is to compare the total steel weight of the unbraced frame system in both connection method, then providing important information in deciding which system to be used in order to achieve economical design in multi-storey unbraced frame.

2. RANGE OF APPLICATION

The range of the study is for two and four bays with heights of two, four, six and eight storeys. In recognition of unlikelihood of the frame consisting of only one longitudinal bay, the minimum number of bays in the out of plane framing was taken as two. Each longitudinal bay was assumed to be 6m in length and all beams assumed to be fully restrained. The limitations on frame dimensions conformed to those specified in the existing guide for wind moment design. The summary of the frame dimension and loading are given in Table 1 and Table 2.

For ultimate limit states, all loadings are in accordance with the values suggested in wind moment design for unbraced frame. The greatest wind load was combined with the minimum gravity load, by choosing appropriate load values and column lengths. Basic wind speeds were taken as the hourly mean speed estimated to be exceeded on average once in 50 years. Wind forces were calculated in accordance with BS6399-2: 1997. Wind forces were considered as horizontal point loads acting on the windward external columns at each floor level. In design, account was taken of the compressive axial forces in the leeward columns, contributed by the horizontal wind. No account was taken of wind uplift on the roof, as this would relieve the compressive axial forces in the columns.

For serviceability limit states, the sway-deflection limit is taken as h_T / 450 for partial strength connections and h_T / 300 for full strength connections, where h_T is the total height of the multi-storey frame.

Table 1. Frame dimension

Scope		Description
Number of bay		2 and 4
Number of storey		2, 4, 6, 8 storeys
Bay width		6m
Longitudinal Bay width		6m
Storey height:	Ground	5m
	Elevated	4m

Table 2. Loading

Gravity Load:		
Dead Load (DL) -	Roof	3.75 kN/m^2
	Floor	3.50 kN/m^2
Live Load (LL) -	Roof	1.50 kN/m^2
	Floor	4.00 kN/m^2
Wind load: basic wind speed		28 m/s

In this study, frames were analyzed under three load combinations as follows:

1. 1.4 dead load plus 1.6 imposed load plus factored notional horizontal force
2. 1.2 dead load plus 1.2 imposed load plus 1.2 wind load
3. 1.4 dead load plus 1.4 wind load

In structural section design, the universal beam sections were used for horizontal members and universal column sections were used for verticals members. All sections were orientated such that loads in the plane of the frame tend to cause bending about the major axis for major axis frame and bending about the minor axis for minor axis frame. All columns are rigidly connected to foundations.

3. PORTAL METHOD OF ANALYSIS

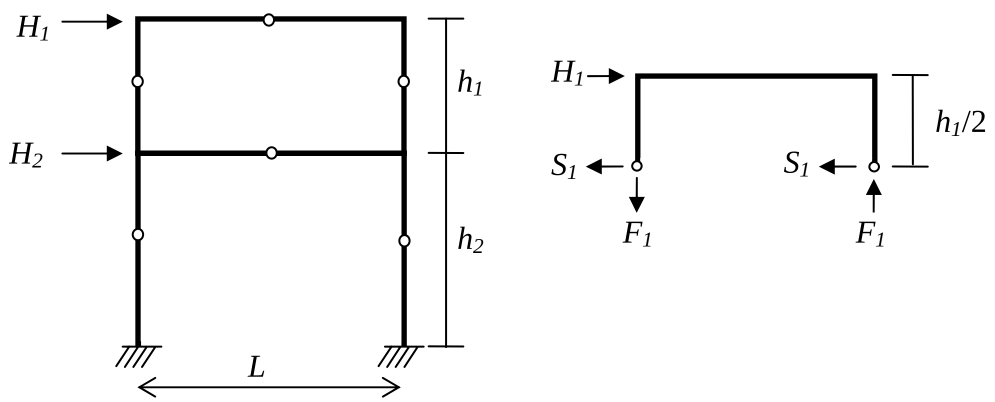

Figure 1 Portal Method of Analysis

In wind moment method analysis, the frame reaction is calculated base on portal method. Referring to Figure 1, each bay of the multi-storey frame is assumed to act as a single portal and the horizontal load:

$$H_1 = \frac{L_1 w_1}{\sum L}$$

The horizontal force is assumed to be divided equally between the two columns on a portion of one bay, thus the shear force:

$$S_1 = \frac{H_1}{2}$$

The vertical forces and moments at the column are therefore:

$$F_1 = \frac{H_1 h_1}{2L}$$

$$M_1 = \frac{S_1 h_1}{2}$$

The internal moment at each end of the beam equals to $M_1 + M_2$. The shear force for the beam is given by:

$$V_1 = \frac{M_1}{L/2} \text{ and } V_2 = \frac{(M_1 + M_2)}{L/2}$$

The portal method analysis simplified the calculation procedures for moment distribution due to horizontal forces. Moments and shear forces obtained from the analysis then combined with the moments calculated from the gravity load. These values then used to design the frames with specific load combination.

4. DESIGN OF MAJOR AXIS FRAME

In the design of major axis frame, it is assuming that the frames are effectively braced at the roof and each floor level to prevent sway about the minor axis of the columns but are unbraced about the major axes columns (see Figure 2 and Figure 3). The prevention of sway about the minor axes can be achieved by cross bracing or by other systems such as attachment to a rigid core.

For beam design, the moment capacity and classification for beam section are in accordance to Steelwork Design Guide to BS 5950: Part 1: 2000 publication of Steel Construction Institute. The floor details are to be such that the beam is effectively restrained against lateral and lateral-torsional buckling. Therefore, no check was done for lateral-torsional buckling. However, when the wind speed is too high while the design of beam is controlled by the moment generated by wind, lateral-torsional buckling should be checked. In this case, *m* is taken to be equal to 0.44 due to the double curvature effect. The studies have shown thought that lateral-torsional buckling is not critical.

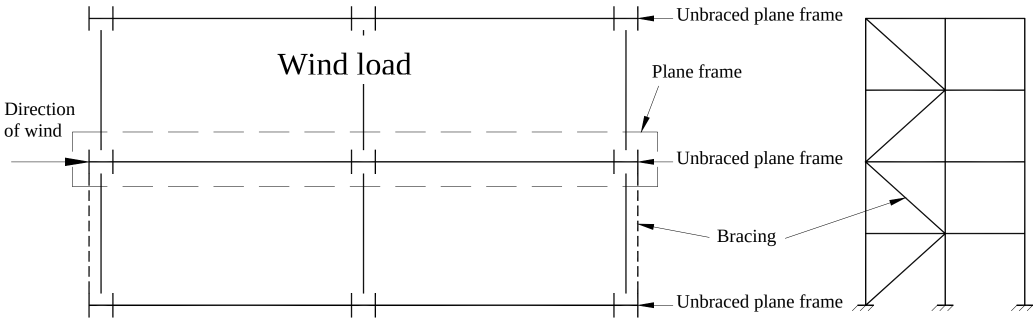

Figure 2 Plane frames braced against out-of-plane sway.

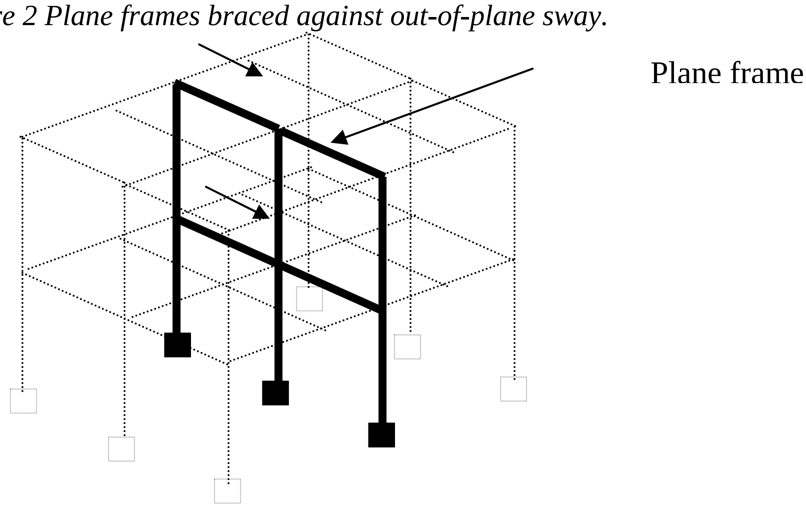

Figure 3 Typical plane frame of two bays two-storey structure.

In column design, the moments in the columns due to vertical load alone are given in the algebraic sum of 10% end restraint moments from the beams and nominal moments due to eccentricity of the beam reactions. Additional internal moments and forces due to horizontal forces (wind load and notional horizontal force) are calculated from the portal method analysis as proposed in wind moment design. The graphical method of Woods is applied to determine the sway-deflection for frames in wind moment design. In wind moment method, the frames were analysed as an elastic rigid-jointed frame, therefore the sway-deflection limit are:

Sway-deflection limit, $\Delta = \dfrac{h_T}{300}$ mm

4.1 DESIGN OF BEAMS

The internal moments and forces in the design of beam are in accordance with the requirements of BS 5950-1: 2000 for simple construction. The beams are assumed to be simply supported and the design normally governed by the maximum sagging moment at the mid length of the beam. Sections used in the design of beam are either plastic or compact sections which in accordance with the recommendation in wind moment method. The design moment (M) should be smaller than the plastic moment resistance (M_c) of the section in order to provide sufficient rotational restraint to the column. The plastic moment resistance is an equation of design strength of the steel (p_y) multiplied by the plastic modulus of the beam section (S). For parts of beams that are effectively unrestrained according to BS 5950-1: 2000, the equivalent uniform moment ($\overline{M}$) should be checked with the lateral torsional buckling resistance moment (M_b). The calculation for equivalent uniform moment is in accordance with the BS 5950-1: 2000 Clause 4.3.5.3.

4.2 DESIGN OF COLUMNS

The sections in the design of columns should be classified as plastic or compact sections. As the frame is unbraced about the major axis but braced on minor axis, the columns are designed to have buckling about the major axis. The frame is a sway frame on major axis; therefore the effective length of the column should be taken as 1.5*L* for major axis and 1.0*L* for minor axis where *L* is the height of the column. The compression resistance and buckling resistance moment are calculated based on the Steelwork Design Guide as mentioned above. The buckling resistance moment is that for "simple" design as stated in BS 5950-1: 2000 Clause 4.7.7. As for column design, the following relationship needs to be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} \leq 1.0$$

where

F_c is the applied axial load due to vertical loading, or a combination of vertical loads and wind loads

M_x is the applied moment about the major axis due to appropriate combination of vertical loading, notional horizontal forces and wind loads

M_y is the applied moment about the minor axis due to appropriate combination of vertical loading

P_y is the design strength of steel

Z_y is the elastic modulus about the minor axis

P_c is the compressive resistance

M_{bs} is the lateral torsional buckling resistance moment for simple design.

4.3 DESIGNS AT SERVICEABILITY LIMIT STATE

The designs at the serviceability limit state consist of various requirements in BS 5950-1: 2000, but horizontal deflection is the only consideration in wind moment method. The deflection limits given in BS 5950-1: 2000 with the purpose to ensure that the resistance and in-service performance of the structure are not impaired. A sensible limit on horizontal deflection for low-rise frames is height / 300. The vertical deflections of beams should generally be calculated using unfactored imposed loads assuming that the beams are simply supported. The limits on imposed load deflection should generally be in accordance with BS 5950-1: 2000 span / 360 for beams carrying plaster or other brittle finishes.

The frames are checked for sway using the unfactored wind loads. Full analysis of frames taking into account connection flexibility shows that partial strength connections deflect significantly more under horizontal loading than those with fully rigid connections. This increased sway can be allowed for by the designer by means of a simple amplification factor applied to the sway deflection. The simple graphical Wood method as recommended in wind moment method sufficiently calculate rigid frame deflections, without taking into account the sway due to asymmetric vertical loads. The calculated rigid frame deflections then increased by 50% as an approximate allowance for the flexibility of partial strength connections. If the deflections are unacceptable, the size of the member will be increased to obtain the required stiffness for frame stability.

5. DATA ANALYSIS AND DISCUSSION

The parametric study on unbraced frame design in full strength and partial strength connection using wind moment method has been carried out. The detailed results are recorded as in Table 3(a and b) for partial strength connections and Table 4(a and b) for full strength connections. The results show that the section designation and the maximum sway using rigid connections are smaller than partial strength connections. Comparison of the total steel weight for full strength connections and partial strength connections is listed in Table 5 (a). From the comparison, it is shown that frames design using full strength connections are lighter than the frame design using partial strength connections. The percentage steel weight saving was in the range of 5.56% to 21.80% for major axis frame. The comparison of sway-deflection is given in Table 6 (a). The sway values for frames in full strength connections are less than frames in partial strength connections. This is due to the deflection limit for partial strength connections has been increased by 50%. To satisfy the serviceability limit state, the size of the steel member is increased in order to attain this sway limit. On the other hand, the design of multi storey unbraced frames under maximum wind speed is control by the serviceability limit state but not the ultimate limit state. For both design approaches, frames with 4 bays acted better against wind load compared to 2 bays frames.

6. CONCLUSION

From the results analysis, several conclusions can be drawn:

1. Wind moment method can be used to design the unbraced steel frames up to 8 storeys with column bending on major axis.
2. For unbraced frame design under minimum gravity load in conjunction with maximum wind load, the design is controlled by serviceability limit state, where the size of the steel members has to be increased to limit the sway-deflection.
3. Frames designed in partial strength connections require stiffer column to resist the horizontal sway deflections. Therefore the size of the column is larger than those required for frames designed in full strength connections.
4. The total steel weight savings for frame using full strength connections compare to partial strength connections is ranging from 5.56% to 21.80% for major axis frames.
5. The design of multi-storey unbraced steel frames using full strength connections performs better than partial strength connections.

Table 3a Wind moment design for major axis frames with partial strength connections.

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	19.90											406x178x60	356x171x45	up to 2nd Floor	203x203x71	254x254x73
2 bay 4 storey	41.10											457x152x67	406x140x46	up to 2nd Floor	305x305x118	305x305x129
												406x178x60		2nd to 4th Floor	254x254x73	254x254x89
2 bay 6 storey	61.14											533x210x92	356x171x45	up to 2nd Floor	356x368x153	356x368x153
		6	5	4	2	6	21.0	24.0	22.5	9.0	28	457x152x82		2nd to 4th Floor	356x368x129	356x368x129
												406x178x60		4th to 6th Floor	254x254x89	254x254x89
2 bay 8 storey	86.81											610x229x113	406x140x46	up to 2nd Floor	356x368x202	356x368x202
												533x210x92		2nd to 4th Floor	356x368x177	356x368x177
												457x152x82		4th to 6th Floor	356x368x153	356x368x153
												406x178x60		6th to 8th Floor	254x254x89	305x305x97

Table 3b Wind moment design for major axis frames with partial strength connections.

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	17.03	6	5	4	2	6	21.0	24.0	22.5	9.0	28	406x178x60	356x171x45	up to 2nd Floor	203x203x52	203x203x52
4 bay 4 storey	41.12											406x178x60	356x171x45	up to 2nd Floor	254x254x89	254x254x89
												406x178x60		2nd to 4th Floor	203x203x52	203x203x52
4 bay 6 storey	66.62											406x178x60	356x171x45	up to 2nd Floor	305x305x118	305x305x137
												406x178x60		2nd to 4th Floor	254x254x107	305x305x118
												406x178x60		4th to 6th Floor	203x203x60	203x203x60
4 bay 8 storey	75.91											610x229x101	356x171x45	up to 2nd Floor	254x254x132	356x368x153
												457x191x98		2nd to 4th Floor	305x305x97	305x305x118
												533x210x82		4th to 6th Floor	254x254x89	305x305x97
												406x178x60		6th to 8th Floor	203x203x60	203x203x60

Table 4a Wind moment design for major axis frames with full strength connections.

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
2 bay 2 storey	19.38	6	5	4	2	6	21.0	24.0	22.5	9.0	28	406x178x60	356x171x45	up to 2nd Floor	203x203x52	203x203x60
2 bay 4 storey	41.15											457x152x67	406x140x46	up to 2nd Floor	254x254x89	305x305x97
												406x178x60		2nd to 4th Floor	203x203x60	203x203x60
2 bay 6 storey	60.90											533x210x92	356x171x45	up to 2nd Floor	305x305x118	356x368x129
												457x152x82		2nd to 4th Floor	254x254x89	305x305x97
												406x178x60		4th to 6th Floor	203x203x60	203x203x60
2 bay 8 storey	87.36											610x229x113	406x140x46	up to 2nd Floor	305x305x137	356x368x153
												533x210x92		2nd to 4th Floor	305x305x118	305x305x118
												457x152x82		4th to 6th Floor	254x254x89	305x305x97
												406x178x60		6th to 8th Floor	203x203x60	203x203x71

Table 4b Wind moment design for major axis frames with full strength connections.

Basic Frame Type	Total Deflection (mm)	Width of Bay (m)	Height of Column		No of Longitudinal Bay	Width of Longitudinal Bays (m)	Gravity Load (kN/m)				Basic Wind Speed (m/s)	Wind Moment Design				
							Floor		Roof			Universal Beam			Universal Column	
			Ground (m)	Elevated (m)			DL	LL	DL	LL		Floor	Roof		External	Internal
4 bay 2 storey	12.93	6	5	4	2	6	21.0	24.0	22.5	9.0	28	406x178x60	356x171x45	up to 2nd Floor	203x203x46	203x203x46
4 bay 4 storey	33.42											406x178x60	356x171x45	up to 2nd Floor	203x203x71	254x254x73
												406x178x60		2nd to 4th Floor	203x203x46	203x203x46
4 bay 6 storey	52.68											406x178x60	356x171x45	up to 2nd Floor	254x254x107	305x305x118
												406x178x60		2nd to 4th Floor	203x203x86	254x254x89
												406x178x60		4th to 6th Floor	203x203x52	203x203x52
4 bay 8 storey	66.65											610x229x101	356x171x45	up to 2nd Floor	305x305x97	305x305x137
												457x191x98		2nd to 4th Floor	203x203x86	305x305x97
												533x210x82		4th to 6th Floor	203x203x60	254x254x73
												406x178x60		6th to 8th Floor	203x203x46	203x203x46

Table 5a Comparison of total designed steel weight for major axis frames between partial strength connections and full strength connections.

Unbraced Frame	**Total Steel Weight per Frame (tonne)**		
	UC	**UC**	**% Difference**
	(full strength connections)	**(partial strength connections)**	
2 Bay 2 Storey	2.736	3.195	14.37
2 Bay 4 Storey	6.795	8.045	15.54
2 Bay 6 Storey	12.433	14.799	15.99
2 Bay 8 Storey	18.551	23.722	21.80
4 Bay 2 Storey	4.590	4.860	5.56
4 Bay 4 Storey	10.489	11.485	8.67
4 Bay 6 Storey	18.984	21.047	9.80
4 Bay 8 Storey	29.709	33.051	10.11

Table 6a Comparison of sway-deflection for major axis frames due to wind load.

Unbraced Frame	**Sway Deflection**			
	UC	**UC**	**Sway Limit**	**Difference**
	(partial strength connections)	**(full strength connections)**	**(h_T/300)**	**(mm)**
2 Bay 2 Storey	19.90	19.38	30.00	-0.52
2 Bay 4 Storey	41.10	41.15	56.67	0.05
2 Bay 6 Storey	61.14	60.90	83.33	-0.24
2 Bay 8 Storey	86.81	87.36	110.00	0.55
4 Bay 2 Storey	17.03	12.93	30.00	-4.1
4 Bay 4 Storey	41.12	33.42	56.67	-7.7
4 Bay 6 Storey	66.62	52.68	83.33	-13.94
4 Bay 8 Storey	75.91	66.65	110.00	-9.26

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