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Diagrid structures for tall buildings: case studies and design considerations

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SUMMARY

The originality of form is one of the new trends that can be identified in the current design of tall buildings. In this design trend, the so-called diagrid structures, which represent the latest mutation of tubular structures, play a major role due to their inherent esthetic quality, structural efficiency and geometrical versatility. In this paper, an overview on application of such typology to high-rise buildings is carried out; in particular, in the first part of the paper, the peculiarities of diagrid systems are described: starting from the analysis of the internal forces arising in the single diagrid module due to vertical and horizontal loads, the resisting mechanism of diagrid buildings under gravity and wind loads is described, and recent researches and studies dealing with the effect of geometry on the structural behavior are discussed. In the second part of the paper, a comparative analysis of the structural performance of some recent diagrid tall buildings, characterized by different number of stories and different geometries, namely the Swiss Re building in London, the Hearst Headquarters in New York and the West Tower in Guangzhou, is carried out, and some general design remarks are derived. Copyright © 2012 John Wiley & Sons, Ltd.

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KEY WORDS: diagrid; steel Structures; optimal diagrid inclination; FEM analysis

1. INTRODUCTION

Diagrids, or exodiagonal systems, are perimeter structural configurations characterized by a narrow grid of diagonal members that are involved both in gravity and in lateral load resistance. Diagonalized applications of structural steel members for providing efficient solutions both in terms of strength and stiffness are not new: earlier examples of diagrid in medium-rise buildings are dated back to the 1960s, with the sketch proposed by Torroja (Figure 1(a)) in his seminal book (Torroja, 1960) and, in the practice, with the 13-story IBM Pittsburgh building (Figure 1(b)), where the 'exterior load bearing truss frame wall of welded steel in a diamond pattern grid was a radical break from post-and-beam construction' and 'gives an unusual liveliness to the façade, after so many years of rectangular curtain-walling.' (Hirschmann, 1965). After this pioneering application of diagrid, the structural designers of tall buildings mainly shifted their attention to another variation of diagonalized systems, the braced or trussed tube, employing mega-diagonal members instead of the narrow grid of diagonal members characteristic of diagrids. On the contrary, nowadays, a renewed interest in and a widespread application of diagrid is registered with reference to large-span and high-rise buildings, particularly when they are characterized by complex geometries and curved shapes, sometimes by completely free forms. Among the large-span buildings, some examples are represented by the Seattle Library (Such, 2005), the London City Hall, the One Shelley Street in Sydney (Wilkinson, 2010) and more recently by several outstanding pavilions realized at the Shanghai 2010 Expo (e.g. France,

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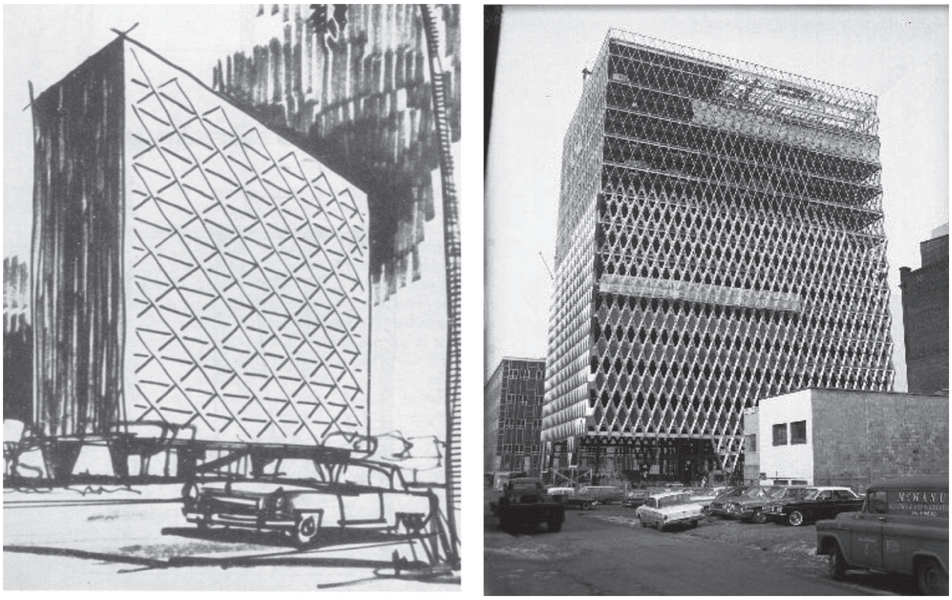

(a) (b)

Figure 1. (a) Sketch of a proposed triangulated wall façade for multistory buildings; (b) the former IBM Pittsburgh building, now United Steel Workers building.

United Arab Emirates), as well as by some dazzling projects such as the Astana National Library (MGS Architecture, 2010) (Figure 2). Among tall buildings, noteworthy examples are the Swiss Re building in London (Munro, 2004), the Hearst tower in New York (Rahimian and Eilon, 2006), the CCTV Headquarters building in Beijing (Carroll *et al*., 2008), the West Tower in Guangzhou (Meng and Qe Zhang, 2006), the Lotte Super Tower in Seoul (Besjak, 2006), the Capital Gate in Abu Dhabi (Schofield, 2012) and the Bow project in Calgary (Charnish and McDonnell, 2008).

With specific reference to tall buildings, diagrids are increasingly employed due to their structural efficiency as well as architectural suggestion. In fact, diagrid structures can be seen as the latest mutation of tube structures, which, starting from the frame tube configuration, have increased structural

Figure 2. Astana National Library (from: http://www.big.dk/projects/anl).

efficiency thanks to the introduction of exterior mega-diagonals in the braced tube solution first suggested by Fazlur Khan in the impressive Chicago John Hancock building; in this case, the significant improvement in terms of lateral stiffness and shear lag reduction also reflects in the building architecture, strongly connoted by the clear and disciplined structure, 'the honesty of structure', in the words of the architect Bruce Graham. The diagrid systems are the evolution of braced tube structures, since the perimeter configuration still holds for preserving the maximum bending resistance and rigidity, while, with respect to the braced tube, the mega-diagonal members are diffusely spread over the façade, giving rise to closely spaced diagonal elements and allowing for the complete elimination of the conventional vertical columns; thus, the diagonal members in diagrid structures act both as inclined columns and as bracing elements and carry gravity loads as well as lateral forces; due to their triangulated configuration, mainly internal axial forces arise in the members, thus minimizing shear racking effects.

In order to assess the behavior of diagrid structures, first of all, the behavior of the elementary triangular unit, appointed as 'diagrid module' as follows, is analyzed under both gravity and lateral loads, and the effect of the module geometry on the structural behavior is briefly discussed.

Then, three significant case studies are examined through the evaluation and comparison of some structural performance parameters.

2. THE TRIANGLE DIAGRID MODULE

The analysis of the diagrid structures can be carried out in a preliminary stage by dividing the building elevation into groups of stacking floors, with each group corresponding to a diagrid module.

As shown in the studies by Moon *et al*. (2007) and Moon (2008), the diagrid module under gravity loads G is subjected to a downward vertical force, $N_{G,\mathrm{mod}}$, causing the two diagonals being both in compression and the horizontal chord in tension (Figure 3(a)). Under horizontal load W, the overturning moment M_W causes vertical forces in the apex joint of the diagrid modules, $N_{W,\mathrm{mod}}$, with direction and intensity of this force depending on the position of the diagrid module, with upward/downward direction and maximum intensity for the modules located on the windward/leeward façades, respectively, and gradually decreasing values for the modules located on the web sides (Figure 3(b)). The global shear V_W causes a horizontal force in the apex joint of the diagrid modules, $V_{W,\mathrm{mod}}$, which intensity depends on the position of the module with respect to the direction of wind load, since the shear force V_W is mainly absorbed by the modules located on the web façades, i.e. parallel to the load direction (Figure 3(c)).

In the formulations provided in Figures 3(a, b, c) for deriving internal forces in the diagrid elements, it has been implicitly assumed that the external load is transferred to the diagrid module only at the apex node of the module itself. However, since the triangle module usually expands over a certain number of stories, transfer of loads to the module occurs at every floor level, and thus also concentrated loads along the diagonal length are present (Figure 4); as a consequence, bending moment and shear force are expected due to this load condition. However, the introduction of a horizontal member at each floor girder to diagonal intersection, an intermediate chord, allows for the absorption of the force component orthogonal to the diagonal direction, thus preserving the prevailing axial force condition. It is worth noting that also in the braced tube system of the John Hancock building, with mega-diagonals and vertical columns, in addition to the main ties restraining the horizontal spread of the X forms, also secondary ties were necessary at each column–diagonal intersection in order to channel loads into vertical columns (Figure 5); this arrangement, as clearly explained in a recent lecture by William F. Baker, let the John Hancock building behave as a tied arch: 'Many people do not realise this, but the John Hancock building is actually a tied arch.' (Baker, 2010).

Furthermore, the above simplified analysis of the diagrid module has been carried out implicitly assuming that the plane of the triangular module coincides with the vertical plane; however, recent applications often concern buildings characterized by curvilinear, non-prismatic forms, which require the study of the diagrid curvature effect on the internal force distribution (Figure 6(a)). In particular, by considering that the single module may be inclined of an angle β with respect to the vertical direction, the effect of both gravity loads and overturning moment gives rise to an additional horizontal force, in the direction orthogonal to the module plane. Therefore, the chords of the diagrid modules,

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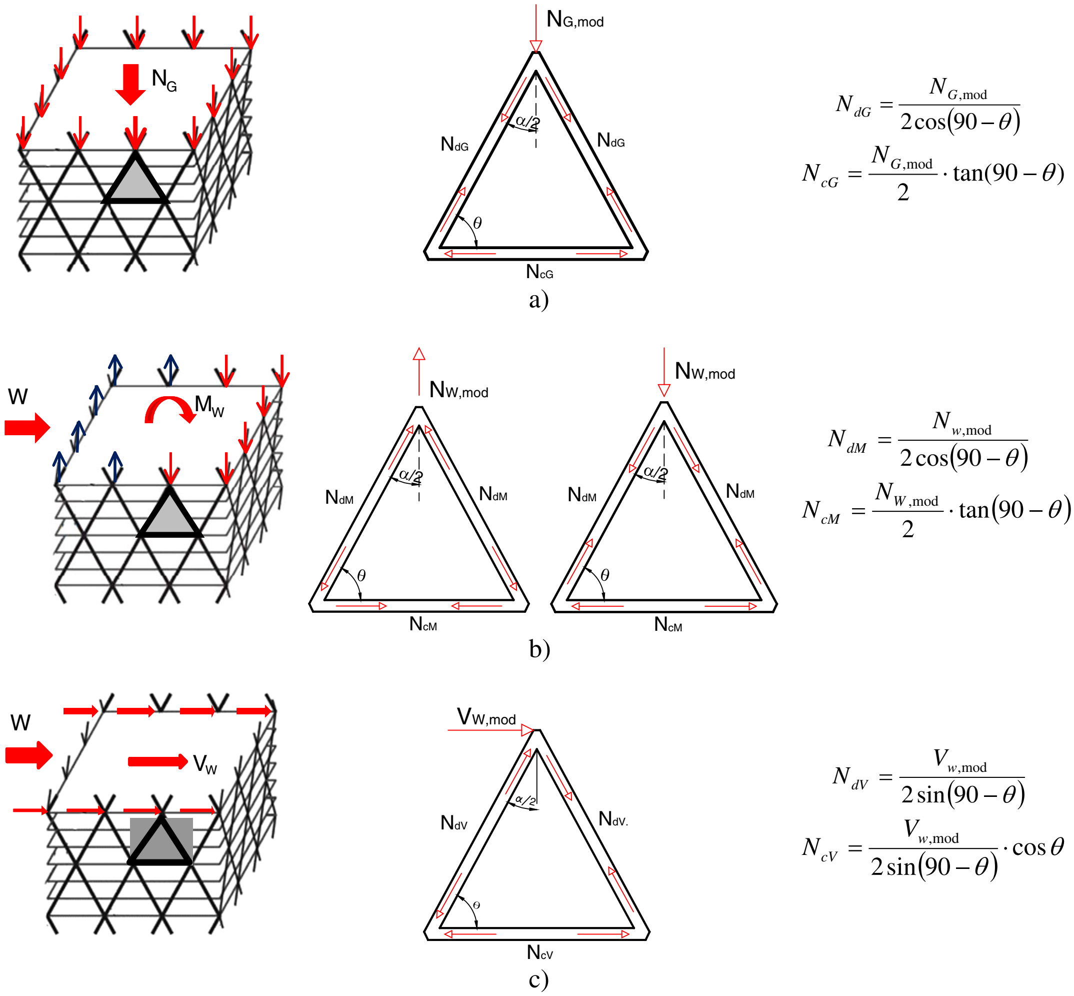

Figure 3. Diagrid module: (a) effect of gravity load, (b) effect of overturning moment and (c) effect of shear force.

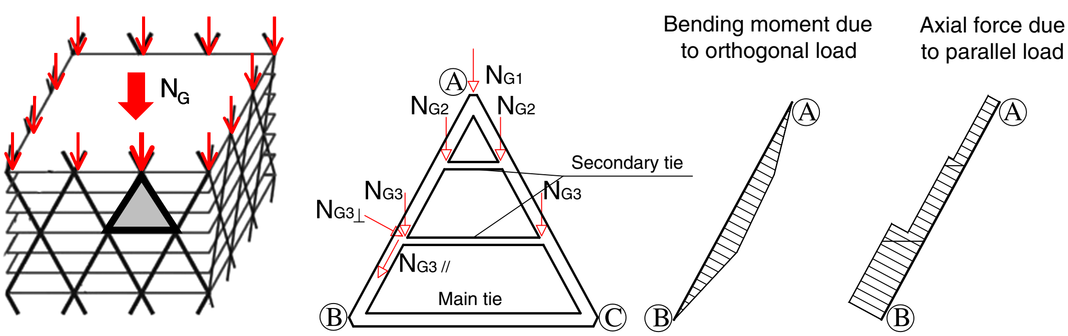

Figure 4. Diagrid module: effect of gravity load along the diagonal length.

continuously connected each other along the building perimeter at the diagonal intersections, also act as hooping elements or ring beams for absorbing these horizontal forces (Figure 6(b, c)). In addition, when the building has non-rectangular, rounded plans, similar effects due to this horizontal curvature develop under the action of lateral shear, and the ring beams also collect these outward forces arising in the horizontal plane (Figure 7).

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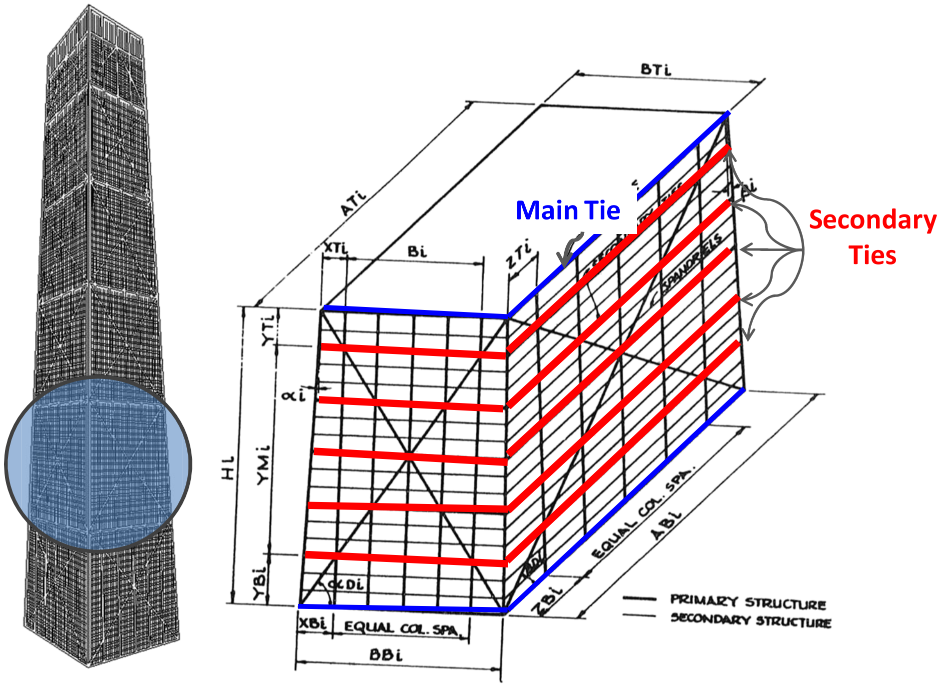

Figure 5. Base tier module of the braced tube system in John Hancock building (redrawn from Khan, 2004).

3. GEOMETRY AND DESIGN CRITERIA

Diagrid structures, like all the tubular configurations, utilize the overall building plan dimension for counteracting overturning moment and providing flexural rigidity. However, this potential bending efficiency of tubular configurations is never fully achievable due to shear deformations that arise in the building 'webs'; with this regard, diagrid systems, which provide shear resistance and rigidity by means of axial action in the diagonal members, rather than bending moment in beams and columns, allows for a nearly full exploitation of the theoretical bending resistance. This is the main reason underlying the extraordinary efficiency of diagrid systems.

Being the diagrid a triangulated configuration of structural members, the geometry of the single module plays a major role in the internal axial force distribution, as well as in conferring global shear and bending rigidity to the building structure. As shown in the study by Moon *et al*. (2007), while a module angle equal to 35° ensures the maximum shear rigidity to the diagrid system, the maximum engagement of diagonal members for bending stiffness would correspond to an angle value of 90°, i.e. vertical columns. Thus, in diagrid systems, where vertical columns are completely eliminated and both shear and bending stiffness must be provided by diagonals, a balance between these two conflicting requirements should be searched for defining the optimal angle of the diagrid module. However, it is worth noticing that, by varying the aspect ratio of the building, the demand for shear and bending stiffness also varies, being slender buildings more governed by a bending behavior than stocky buildings; therefore, it is expected that by increasing the building slenderness, also the optimal angle of the diagrid module should increase. Some useful indications on optimal angle values for buildings characterized by different aspect ratio are provided in the studies by Moon *et al*. (2007) and Moon (2008) and reported in the diagram of Figure 8, where the top displacement of buildings from 20 to 60 stories is depicted as a function of the diagrid angle; on the basis of these results, in Figure 9, the optimal angle values are represented as a function of the number of stories (aspect ratio), showing the expected increase with the building height.

Furthermore, for very tall buildings, i.e. buildings with aspect ratio of the order of 7 or more, the relative demand for shear and bending stiffness is not uniformly distributed along elevation, and a varying-angle diagrid configuration, with steeper angles towards the base, generates more efficient

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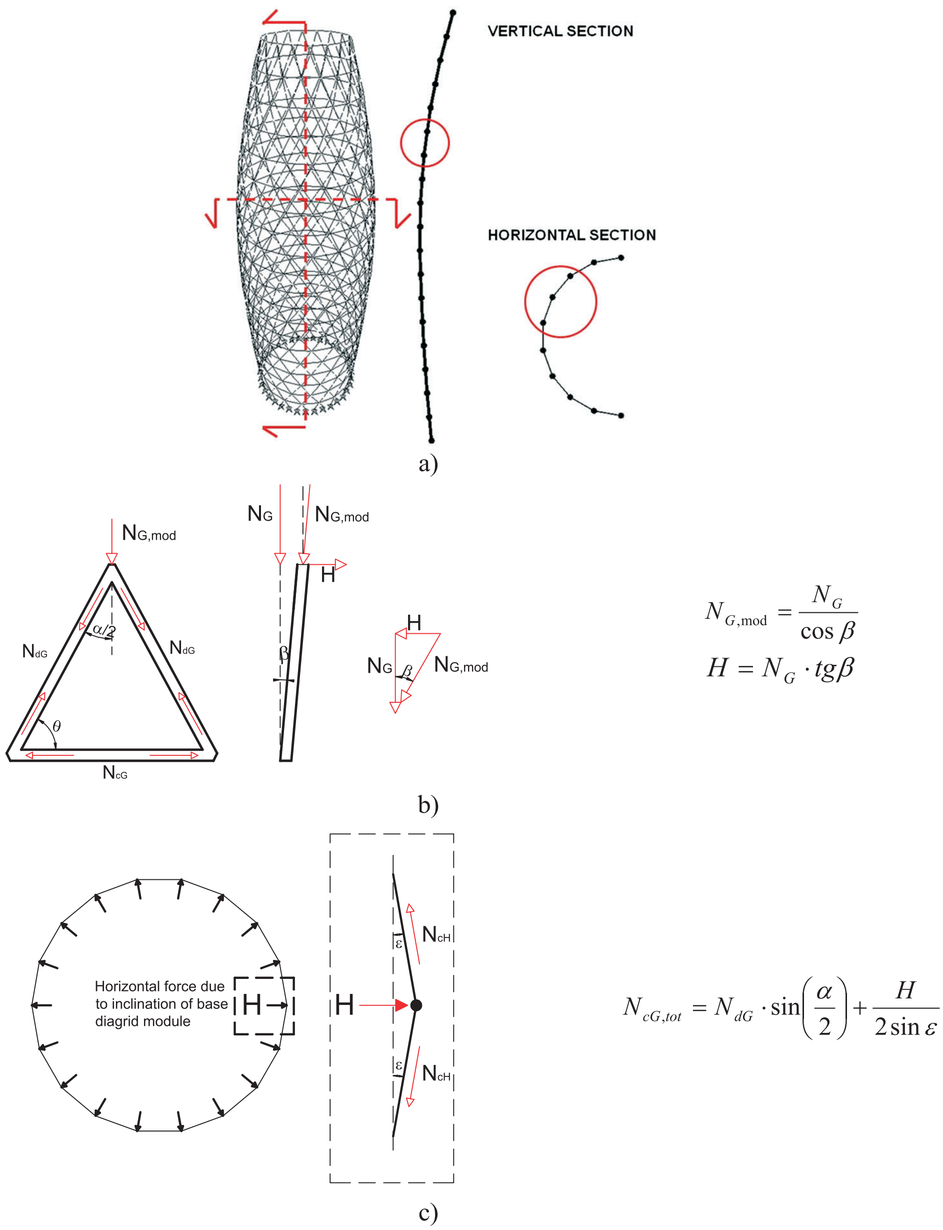

Figure 6. Diagrid module under vertical load—effect of vertical and horizontal curvature.

design solutions (i.e. less material consumption) than uniform angle configurations (Moon, 2008; Zhang *et al*., 2010). In Figure 10, the results of the study by Zhang *et al*. (2010) are reported in a chart format, which provides the optimal values of angle couples (θ_1 at the top and θ_2 at the base) versus the number of stories: it is interesting to notice that the θ_1 and θ_2 angles are coincident for 30-story buildings while significantly diverge in the case of larger number of stories.

4. CASE STUDIES

As follows, some recent diagrid tall buildings, namely the Swiss Re building in London, the Hearst Headquarters in New York and the West Tower in Guangzhou (Figure 11) are briefly presented and

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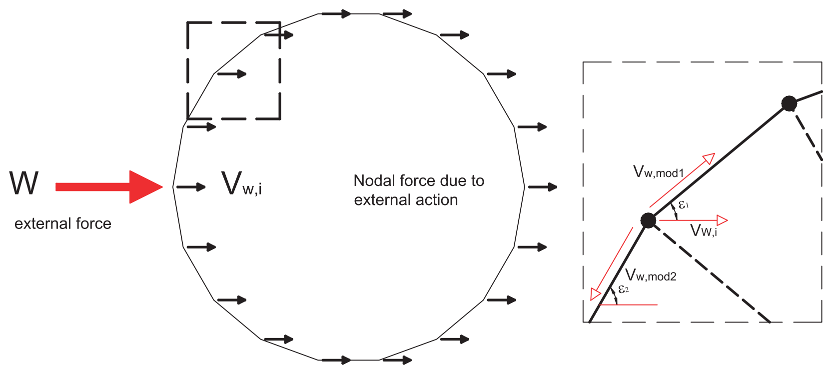

$$V_{w,\mathrm{mod}1} = V_{w,i} \cdot \left(\cos\varepsilon_1 + \frac{sen\varepsilon_1}{tg(\varepsilon_2 - \varepsilon_1)} \right) \qquad V_{w,\mathrm{mod}2} = V_{w,i} \cdot \left(\frac{sen\varepsilon_1}{sen(\varepsilon_2 - \varepsilon_1)} \right)$$

Figure 7. Diagrid module under horizontal load—effect of horizontal curvature.

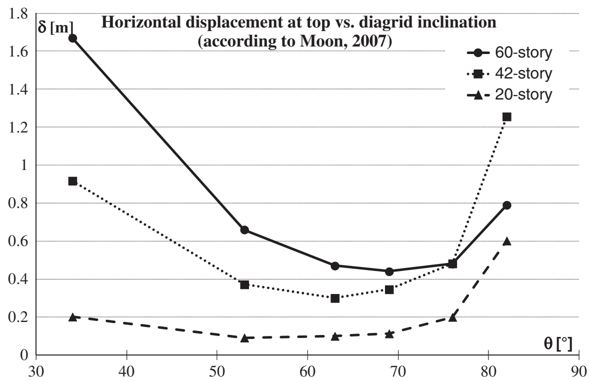

Figure 8. Building top displacement versus diagrid angle (redrawn from Moon *et al*., 2007).

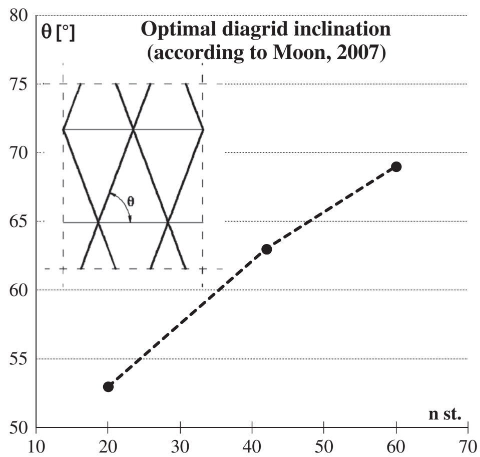

Figure 9. Optimal diagrid inclination for different building heights.

examined in comparative terms. In Table 1, the major building data are provided, while in the next sub-paragraphs, some additional information on the structural system is presented for the three case studies. Data and information herein discussed and adopted for developing structural models of the building structures are derived from journal papers, reports and websites: namely, for the Swiss Re

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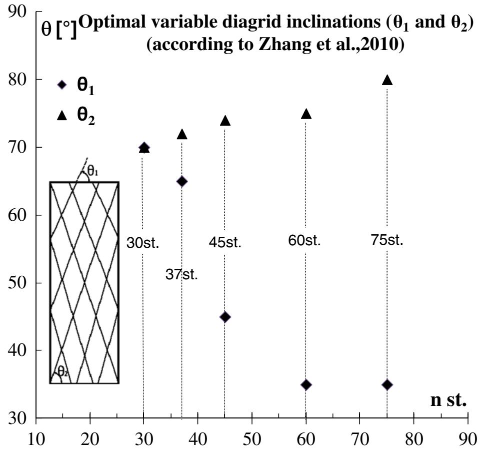

Figure 10. Optimal couples of angles for variable—inclination diagrid.

Figure 11. Case studies: Swiss Re (top left), Hearst Tower (down left) and Guangzhou West Tower (right).

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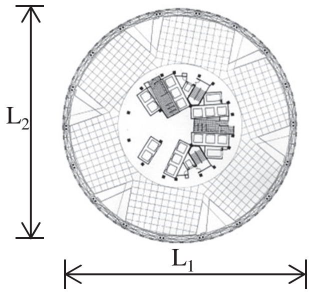
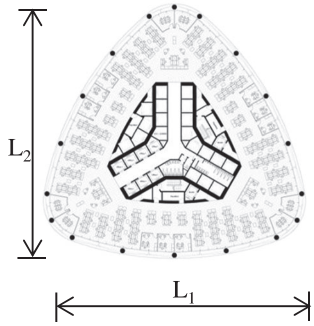
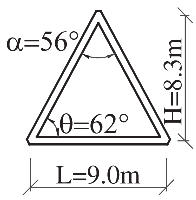
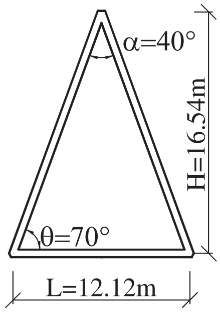
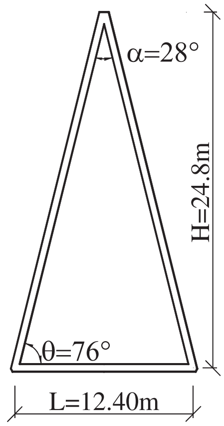
Table 1. Comparison among the three case studies—major data.

	Swiss Re Tower	Hearst Tower	Guangzhou West Tower
Story nos	40	46	103
H [m]	180	183	440
Plan shape			
L_1 [m]	30–56	48	43.5–65
L_2 [m]	30–56	37	43.5–65
H/L_{max}	3.21	3.81	6.77
H/L_{min}	6.00	4.94	10.11
$H/L_{average}$	3.43	-	8.11
A_{tot} [m^2]	74 300	79 500	285 000
$A_{fl.,max}$ [m^2]	2476	1730	3074
$A_{fl.,min}$ [m^2]	1885	1730	1580
A_{core} [m^2]	475 $A_c/A_{max} = 19\%$ $A_c/A_{min} = 25\%$	300 $A_c/A = 17\%$	880 $A_c/A_{max} = 29\%$ $A_c/A_{min} = 55\%$
Span [m]	$d_{max} = 31$ $d_{min} = 5$	$d_{max} = 24$ $d_{min} = 12$	$d_{max} = 16.6$ $d_{min} = 6.8$
Diagrid base module			
Steel wt [t]	8358	10 480	51 310*
Diagrid wt [t]	2423	3040*	14 880*
Steel unit wt [kN/m^2]	1.12	1.32	1.8
Diagrid unit wt: [kN/m^2]	0.32	0.38	0.52

*Evaluated by the authors.

building, Munro (2004) and Plank (2005); for the Hearst Tower, Rahimian and Eilon (2006), Rahimian and Eilon (2008) and Fortner (2006); for the West Tower, Meng and Qe Zhang (2006) and Huang *et al*. (2010).

In the next paragraph, a comparative analysis of the structural behavior under gravity and wind load is carried out for the three buildings, both by means of 'hand calculations' based on the formulae provided in paragraph 2 and by means of finite element method (FEM) computer modeling. Some conclusive remarks on the structural efficiency of diagrid structures, as well as on the accuracy of simplified assessment of the structural behavior, are finally derived.

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4.1. Swiss Re building

30 St Mary Axe—also known as the Swiss Re building—in London (Figure 11, top left) is the first modern application and the most representative example of a diagrid structure. Designed by Sir Norman Foster and by the structural engineer Dominic Munro of Arup, it received the Royal Institute of British Architects Stirling Prize in 2004. The building has 40 stories, globally 180 m tall, with typical interstory height equal 4.15 m; it is circular in plan with diameter changing along the elevation, equal to 56 m at its widest point, at the 20th story, reducing to 49 m at ground level and to 30 m at the 38th level, where a steel and glass dome tops off the building. The double curvature of the building façade, both in the horizontal plane and along the vertical direction, gives rise to the effects that have been discussed in paragraph 2. The diagrid structure is generated by a pattern of intersecting diagonals that follow the helical path of the so-called 'light wells' created for enforcing natural light and air circulation; the steel triangles are two-story high (8.30 m) and 9 m wide, with an intermediate tie connecting the two diagonals, which gives the module the aspect of an 'A-shape frame' (Figure 12). The diagonals are circular hollow section members, with cross section varying between 508×40 mm at the lowest floors and 273×12.5 mm at the top, while the chord members are rectangular hollow sections (RHS) $250 \times 300 \times 25$ mm. The steel–glass dome is constructed as a welded grillage of RHS members ($110 \times 150 \times 8$ mm). The circular central core, which has a constant diameter along the elevation, equal to 25 m, does not contribute to the lateral resistance and rigidity, being a simple frame structure.

4.2. Hearst Headquarters Tower

Also, the Hearst Tower in New York was designed by Sir Norman Foster with the structural engineer firm WSP Cantor Seinuk of New York City; it was the first skyscraper to break ground in New York

Figure 12. Swiss Re: the A-shaped diagrid module (from http://www.30stmaryaxe.co.uk/images/construction/).

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City after 11 September 2001 and received the 2006 Emporis Skyscraper Award, as the best skyscraper in the world completed that year, and the 2008 International Highrise Award. The building, 46 stories and 183 m tall, has a prismatic form and a rectangular floor plan 48 × 37 m and is built on an existent historic six-story building. The diagrid, creating the characteristic 'diamond effect' in the façade, rises from a 10-story structure, made of 12 composite columns and 10 mega-diagonals, all with hollow box sections 1100 × 1100 × 10, filled by C45/55 concrete (Figure 13). The diagrid module is 12.25 m wide and 16.54 m high and covers four stories; the diagonal cross section are I shape, with maximum size W14 × 370 (i.e. depth 455 mm, flange width 419 mm, flange thickness 68 mm and web thickness 42 mm) at the base of the diagrid (10th level), while minimum size at the top is W14 × 132 (i.e. depth 373 mm, flange width 373 mm, flange thickness 26 mm and web thickness 16 mm).

4.3. Guangzhou West Tower

The Guangzhou West Tower has been designed by Wilkinson Eyre architects and by the Arup structural engineer Craig Gibbons; with 103 stories and a total height of 440 m, it is the tallest building in China and one of the 10 tallest in the world; it received the CTBUH 2011 Best Tall Building Award for the Asia & Australasia Region. The building has a curvilinear shape along the elevation, and the floor plate is an equilateral triangle with round corners, with each side 60 m at the base, increasing to a maximum value of 66 m at approximately 1/3 of the way up the building, at which point the side begins to reduce, up to 43.5 m, at the top. It has a composite structure made of a central concrete core and perimeter diagrid structure, with the diagrid module expanding over six stories, 12.4 m wide and 24.8 m high. The diagonals are steel tubular members filled by high strength concrete (60 MPa), with size ranging between 1080 × 55 mm at the first floor and 700 × 20 mm at the top. The concrete core

Figure 13. Hearst Tower: mega-columns and mega-diagonals at first 10 floors (from http://static.worldarchitecturenews.com/project/).

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has a triangle shape with chamfered corners and fully participates to the lateral resistance up to the 70th floor; starting from this level, the core is eliminated, leaving place to a central giant atrium for the hotel that occupies the upper floors (Figure 14).

5. STRUCTURE MODELING

In order to assess the structural behavior of the three buildings under gravity and wind loads, both the formulae provided in paragraph 2 and SAP 2000 (Computers and Structures, Inc. (CSI), Berkley, California, USA) finite element models of the diagrid structures have been utilized.

Some assumptions have been made in the modeling and analysis phases due to the lack of some data concerning the geometry, the structural properties and the loading values.

Concerning the Swiss Re building, while the diagrid cross sections at the lowest and highest levels have been derived from the inherent bibliography (Munro, 2004), no specific information on the member variation along the height was available; for this reason, a step-wise linear variation has been assumed for the diagonal cross sections, according to the sketch provided in Figure 15. Concerning the ring beams, the same cross section has been adopted throughout the diagrid elevation. The 22-m tall steel–glass dome is not explicitly included in the structural model and is considered as an additional dead load equal to 1978 kN acting on the top perimeter diagrid.

Similar assumptions have been made for the Hearst Tower: in particular, the variation of diagonal member sections reported in Figure 16 has been adopted in the structural model, with cross sections at the lowest level of the diagrid (the 10th level of the building) and at the top being derived from bibliography. The 10-story structure underneath the diagrid is explicitly included in the model, with 12 mega-columns and 10 mega-diagonals, all realized through concrete-filled-steel-tube sections; the

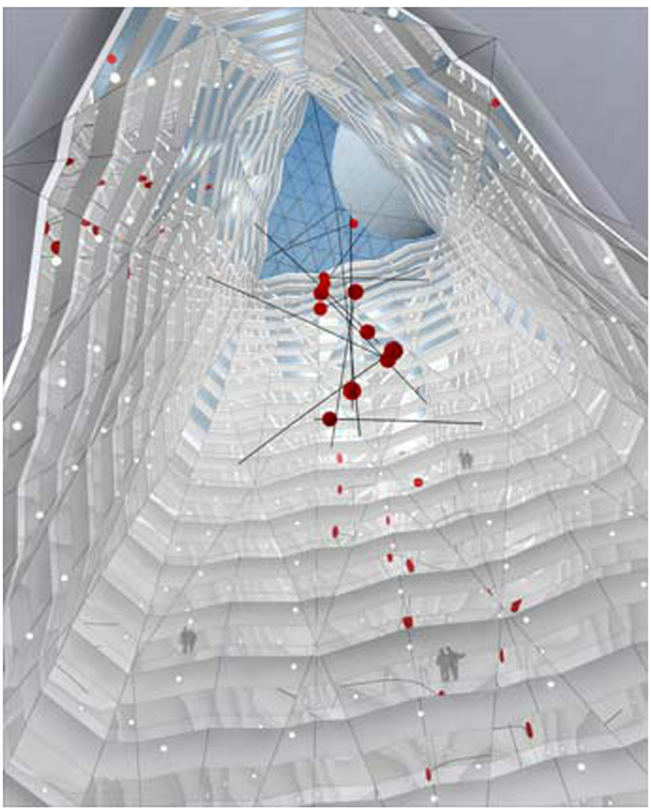

Figure 14. Guangzhou West Tower: top atrium (from http://archrecord.construction.com/ar_china/newsImages/).

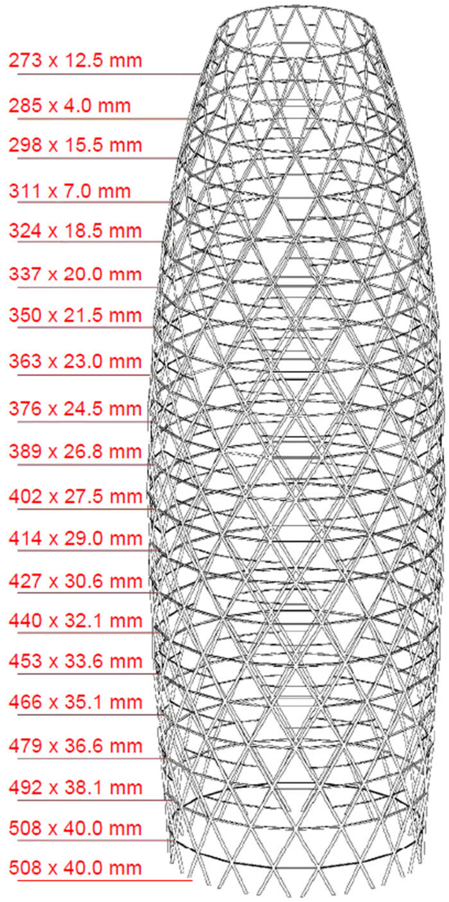

Figure 15. Swiss Re: diagonals cross sections adopted in the FEM model.

mega-diagonals (four in transversal direction and six in longitudinal direction) connect the outer mega-columns at the third floor to the inner core columns at the 10th floor.

Finally, with reference to the Guangzhou West Tower, very few data were available from the literature (Shen *et al*., 2009; Huang *et al*., 2010); also in this case, a step-wise variation of the diagrid member sections has been assumed, as illustrated in Figure 17, starting from the two known values at the first floor and at the top, respectively. Furthermore, the hypothesis of lateral resistance equally shared by the core and the diagrid systems up to the seventieth floor has been made.

6. LOAD VALUES

It is worth underlining that, with the exception of the Swiss Re building, no details on the floor structural system and on the dead load were available; however, considering that the three building plans show similar values of the floor span from the core to the perimeter structure (Table 1), the dead load unit value reported in the '30 St Mary Axe Specifications' (Munro, 2004) has also been adopted for the Hearst and Guangzhou West Towers (Table 2).

Live loads have been derived from building codes for the relevant occupancies identified in the three buildings (office, store, hotel, mechanical space, etc.); in particular for the Swiss Re and Guangzhou Towers, reference has been made to Eurocode 1 (UNI ENV 1991-2-1, 2004), while for the Hearst Tower, the code ASCE 7-05 (ASCE 7-05, 2006) has been considered.

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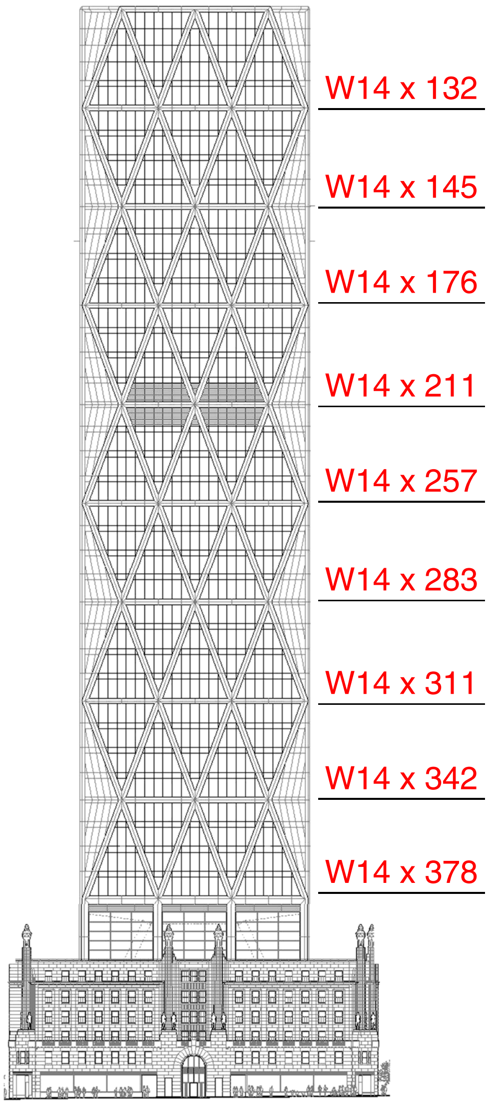

Figure 16. Hearst Tower: diagonals cross sections adopted in the FEM model.

The same codes have also been adopted for computing the equivalent static wind action, starting from a reference wind velocity of 20 m/s for the Swiss Re and the West Tower and of 40.25 m/s for the Hearst Tower.

In Table 2, the values of dead, live and wind loads are provided; for the wind action, both the values of the global base shear and of the overturning moment are provided.

7. ANALYSIS RESULTS

Both the formulae provided in paragraph 2 and the finite element models of the diagrid structures have been utilized for assessing the structural behavior of the three buildings under gravity and wind loads.

As follows, some comparisons between linear FEM analysis results and the 'hand calculation' results are given for the three case studies in diagram format. In particular, the diagrams provide

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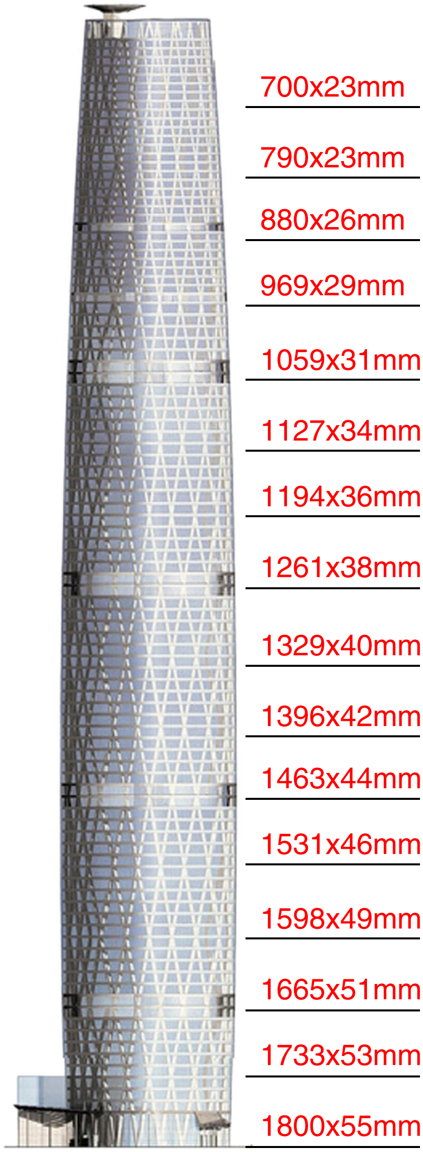

Figure 17. Guangzhou West Tower: diagonals cross sections adopted in the FEM model.

Table 2. Loads assumed in the building analysis.

	Swiss Re Tower	Hearst Tower		G. West Tower
Dead load [kN/m^2]	4.45 (Munro, 2004)	4.45 (assumed)		4.45 (assumed)
Live load [kN/m^2]	Office: 3.00 [EC 1] Store: 5.00 [EC1]	Office: 2.40 [ASCE 7-05] Hall space: 4.80 [ASCE 7-05] Mechanical space: 6.00 [ASCE 7-05]		Office: 3.0 [EC1] Hotel: 5.00 [EC1] Mechanical space: 6.00 [EC1]
Code	Eurocode 1	ASCE 7-05		Eurocode 1
Wind base shear [MN]	31	x: 10	y: 14	140
Wind overturning moment [MN m]	2798	x:1042	y:1377	33 234

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- diagonal axial forces along the height due to gravity load, (Figures 18(a), 19(a) and 20(a));
- diagonal stress level (i.e. axial demand to capacity ratio, DCR) along the height due to gravity load (Figures 18(b), 19(b) and 20(b));
- diagonals axial load at the base of the diagrid structure due to combination of gravity and wind loads (Figures 18(c), 19(c) and 20(c));
- diagonals stress level (i.e. axial DCR) at the base of the diagrid structure due to combination of gravity and wind loads (Figures **18**(d), **19**(d) and 20(d));
- lateral displacements along elevation for the three case studies (Figure 21).

For the Hearst Tower, two orthogonal wind directions have been considered (Figures 19(c, d, e, f)) since the rectangular plan features two different façades; on the contrary for the Swiss Re building and Guangzhou West Tower, both characterized by axial–symmetrical plan, only one wind direction has been considered.

With reference to the analysis results, a preliminary thorough consideration concerns the comparison between the hand calculation and FEM analysis: it is worth noticing that all the graphs reporting the internal forces in the diagrid prove a very good correspondence between FEM analysis and hand calculation.

By analyzing in detail the results reported in the first series of graphs, i.e. the distribution along the height of gravity compressive forces in the diagonal members (Figures 18(a), 19(a) and 20(a)), the same linear distribution for the three buildings can be observed, as trivially expected. Further, the maximum diagonal forces arising at the base of the Swiss Re building and of the West Tower are of two different orders of magnitude, i.e. around 8.5 MN and 70 MN, respectively; also, this result can be quite trivially explained by comparing the values of total floor area, i.e. 74.300 m^2 versus 285.000 m^2, and the number of diagonals at the base, 36 versus 25, respectively, for the Swiss Re building and the West Tower. Finally, it is worth

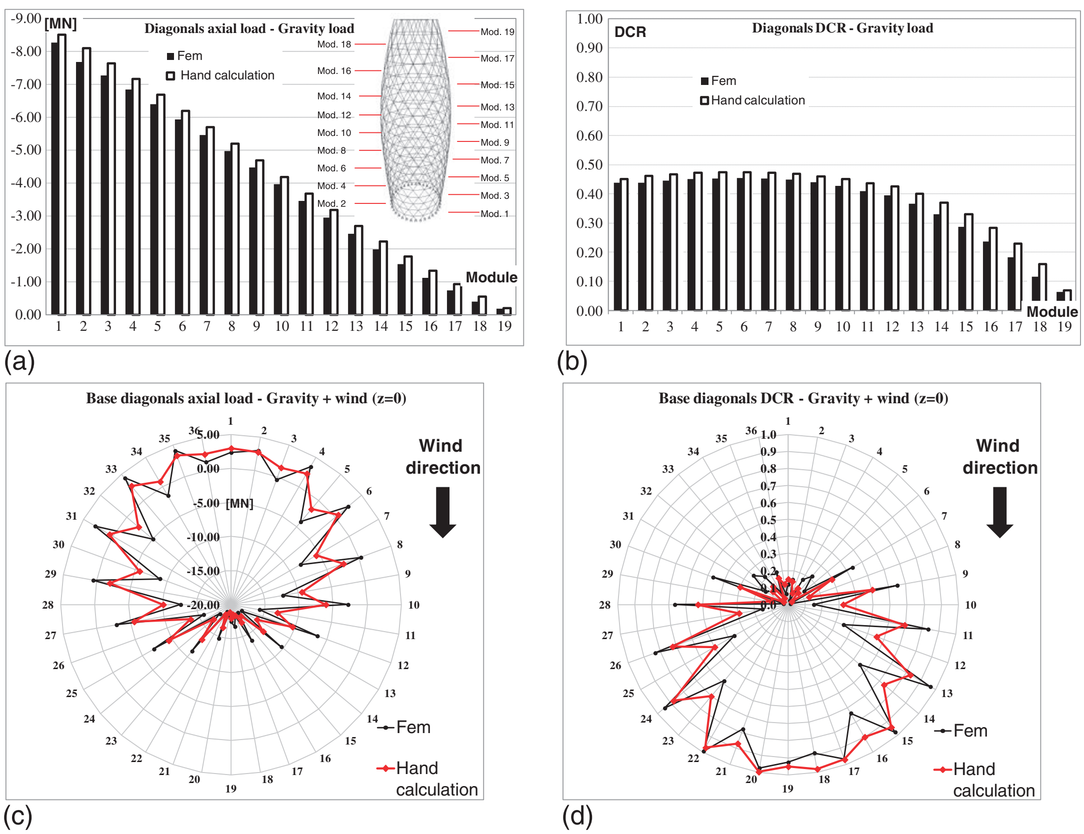

Figure 18. Swiss Re building: (a) diagonals axial load due to G, (b) diagonals stress level due to G, (c) diagonals axial load due to $G + W$ and (d) diagonals stress level due to $G + W$.

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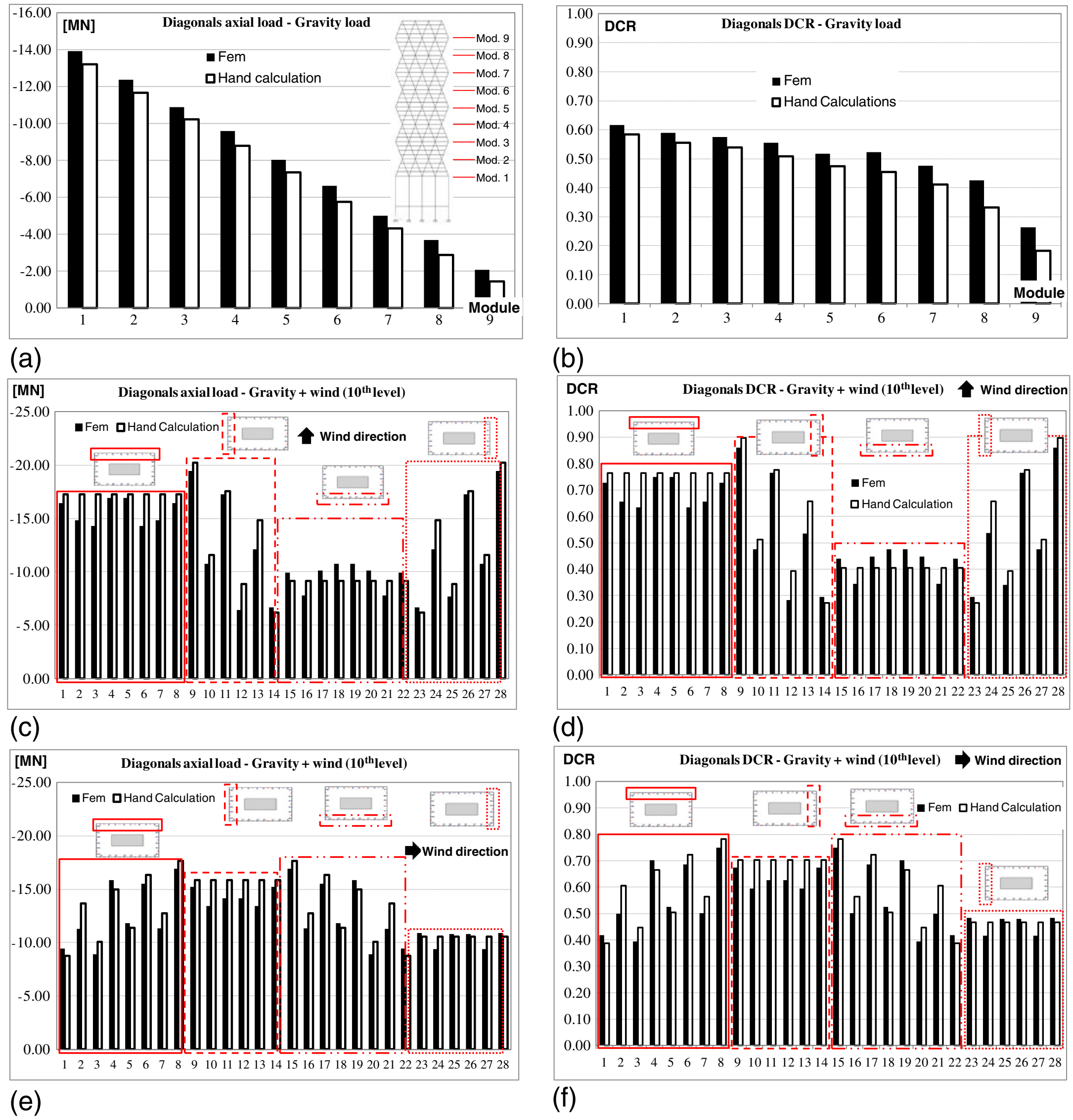

Figure 19. Hearst tower: (a) diagonals axial load due to G, (b) diagonals stress level due to G, (c) diagonals axial load due to $G + W$ (y direction), (d) diagonals stress level due to $G + W$ (y direction), (e) diagonals axial load due to $G + W$ (x direction) and (f) diagonals stress level due to $G + W$ (x direction).

noticing that the maximum value of diagonal axial force in the Swiss Re building, 8.5 MN, is well in agreement with the value provided in the inherent bibliography.

In the charts of Figures 18(b), 19(b) and 20(b), where the diagonal DCR (i.e. the compressive axial force in the diagonal members divided by the relevant buckling resistance) is provided along the height of the buildings, an interesting consideration can be made: the three structures under gravity load show similar DCR values, ranging between 0.4 and 0.6, and all present an almost uniform distribution along the height, with lower values only at very top levels; in particular for the Swiss Re building, the gravity DCR is around 0.45 and only at the highest levels decreases, for the Hearst Tower, it is comprised between 0.40 and 0.60 and for the West Tower, it is almost uniformly equal to 0.45. However, under combined gravity plus wind loads (Figures 18(c, d), 19(c, d, e, f) and 20(c, d)), a more complex distribution of axial load in diagonals is obtained, but the result systematically observed in the three cases is that the DCR values for the diagonals located on the leeward façades, at diagrid base, are very close to 1.0.

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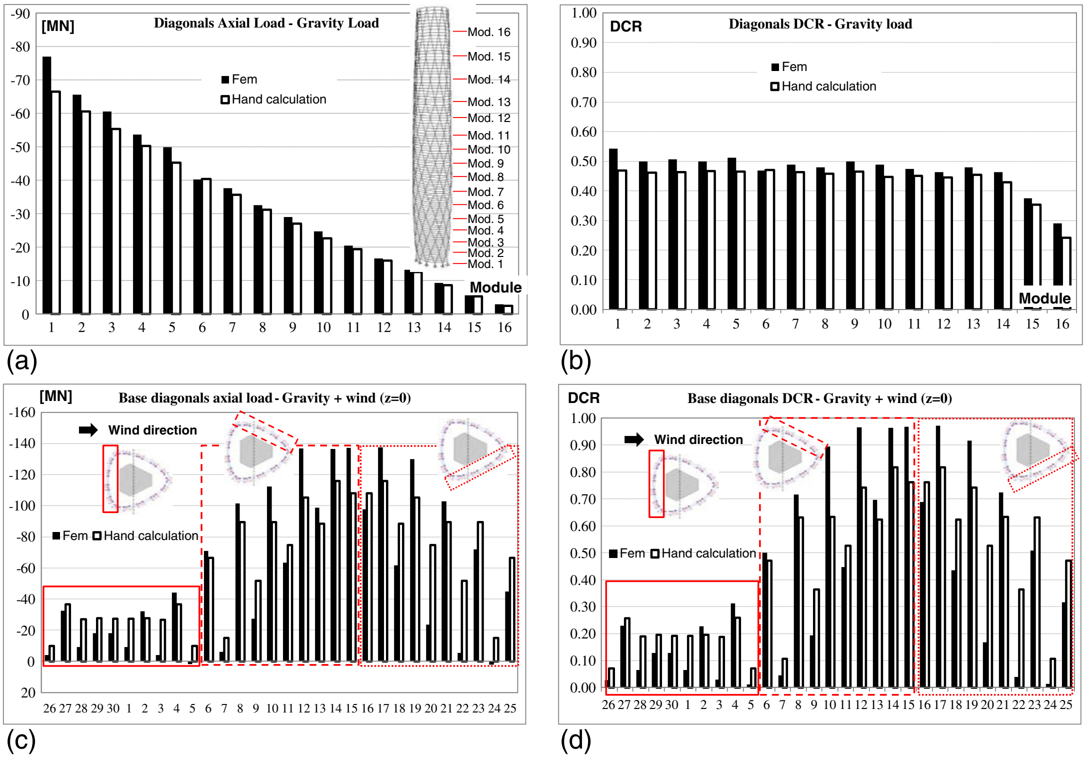

Figure 20. Guangzhou West Tower: (a) diagonals axial load due to G, (b) diagonals stress level due to G, (c) diagonals axial load due to $G+W$ and (d) diagonals stress level due to $G+W$.

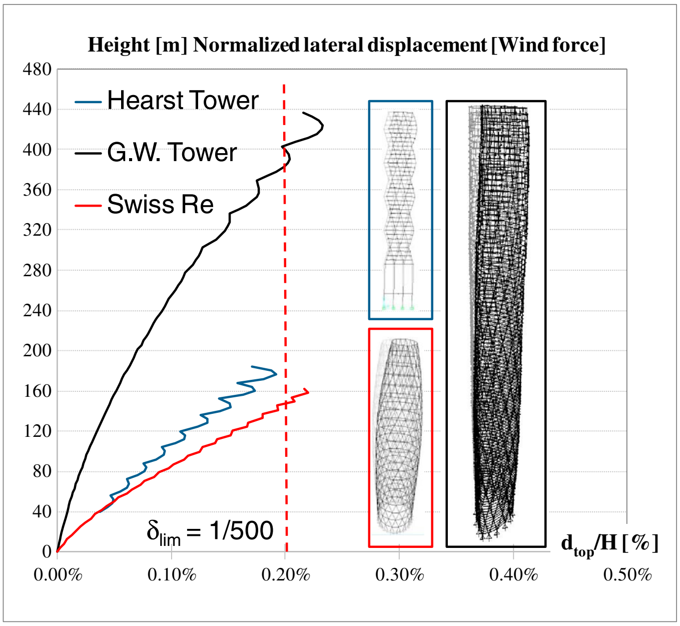

Figure 21. Normalized lateral displacement under wind force.

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The deformation of the structures under gravity plus wind loads has also been assessed from the FE analyses; as can be observed from Figure 21, the three towers assume a deformed configuration, which suggests the prevailing cantilever behavior, thus confirming the significant reduction of the racking deformation component thanks to the diagrid arrangement. Further, the top drift ratios are very close to 1/500, which is the deformation limit usually adopted in tall building design practice, thus confirming the high stiffness of this structural typology.

The dynamic behavior of the buildings has also been assessed showing a very regular response, with the first two vibration modes mainly involving translation in the two plan orthogonal directions, while torsional behavior is only associated to the third mode, characterized by a significant lower period than the first two ones. The higher modes follow the same sequence as of the first three modes (i.e. translation in the principal plan direction, translation in the other principal plan direction and torsion).

8. DISCUSSION ON ANALYSES RESULTS AND DESIGN IMPLICATIONS

By adopting a framed tube system as a term of comparison, namely the World Trade Center (WTC) structure, which has been deeply analyzed by the authors, the dramatic efficiency of the diagrid system can be even better appreciated; in fact, from the analyses carried out by De Luca *et al*. (2003) and also confirmed by the comprehensive results obtained by NIST in the extensive post-WTC-collapse research (Sadek, 2005), some interesting considerations can be extracted.

The maximum values of the DCR in the base columns of the WTC framed tube under gravity plus wind loads were around 0.4–0.7 (De Luca *et al*., 2003; Sadek, 2005); this clearly suggests that strength was not the governing criterion in sizing the column cross sections. Notwithstanding, the lateral response of the WTC structure shows values of cumulative drift under original design wind loads in the range of H/300–H/200 (Table 3), quite larger than H/500; as a comment to the above results, in the NIST report, it is stated that '... limitation of total building drift under wind loads was not part of the original WTC design criteria' (Sadek, 2005).

By putting together these two response features of the WTC framed tube, i.e. quite limited column stress level (0.4–0.7) and quite large top drift (H/300–H/200), a certain inefficiency of the frame tube structural type does emerge; on the contrary, the high values of diagonal DCR and top drift in the range of H/500, which have been thoroughly registered for the three analyzed buildings, testify the great efficiency of diagrid systems.

Some general conclusive remarks arise from (i) the assessment of the structural behavior, and (ii) the possibility of analyzing diagrid buildings, in a very preliminary phase, through simple formulae.

Concerning the structural behavior, the three diagrid buildings examined in this paper show similar values of stress level in the diagonals (DCR), both under gravity loads (around 0.40–0.60) and under gravity plus wind loads (close to 1.0). These values confirms that, thanks to the high rigidity of the diagonalized façade, the sizing of the steel members is mainly governed by strength criteria; as a matter of fact, the total wind sway is close to the limit of H/500, quite universally assumed as a threshold value in the design practice.

The systematic observation of this response in the three case studies, significantly different in terms of height, elevation shape and plan geometry, underlines that the diagrid structures have an optimum

Table 3. Lateral deformation parameters of the WTC frames tube under wind loads (redrawn from Sadek, 2005).

Loading case	WTC 1			
	E–W		N–S	
	Total drift (in.)	Drift ratio	Total drift (in.)	Drift ratio
Original design case	56.6	H/304	55.7	H/309
SOP case	56.8	H/303	68.1	H/253
Refined NIST case	70.6	H/244	83.9	H/205

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behavior, provided that the unit module is configured according to a suitable angle; in fact, the optimal values suggested by Moon *et al*. (2007) and provided in Figure 9 are quite close to the values of the module angle adopted in the three buildings, as can be observed in the chart of Figure 22.

The high lateral stiffness, as well as the high torsional rigidity deriving from the tubular configuration of the building (i.e. the perimeter position of the diagrid system), also ensures a very good level of overall dynamic performance. Other analysis results not reported in this paper suggest additional advantages of diagrid structures mainly related to the high redundancy and resistance to progressive collapse. Such desirable performance is associated with a low unit steel weight of the diagrid system (in the range of 30–50 kg/m^2 for the three buildings; Table 1), which confirms the great structural efficiency.

All the above considerations, coupled with the possibility of adapting the diagrid to nearly every building shape and of obtaining elegant façade appearances through an integrated architectural–structural design, are the main reasons for the increasing popularity of the diagrid systems.

Finally, the simplicity and straightforwardness of the structural system, made of triangulated frame units with members mainly working in axial force condition, allow for simplified 'hand analysis' of the building with a very good approximation degree.

9. CONCLUDING REMARKS

In this paper, an overview on the structural behavior of diagrid structures in tall buildings has been provided. Starting from the evaluation of internal forces arising in the single triangle module under the effects of both gravity and wind loads, a discussion on the effects of the building form as well as of the diagonal slope has been presented. The above considerations on the assessment of internal forces have been applied to three case studies, namely the Swiss Re building in London, the Hearst Headquarters in New York and the West Tower in Guangzhou, and the results have been compared with analogous results obtained through computer analyses.

The following observations and remarks can be stated on the basis of the building analysis results:

- the analyses show equal stress level in the diagonals (i.e. DCR around 0.40–0.60 under gravity loads and close to 1.0 both under gravity and wind loads);
- the structural design of the steel members is mainly governed by strength criteria;

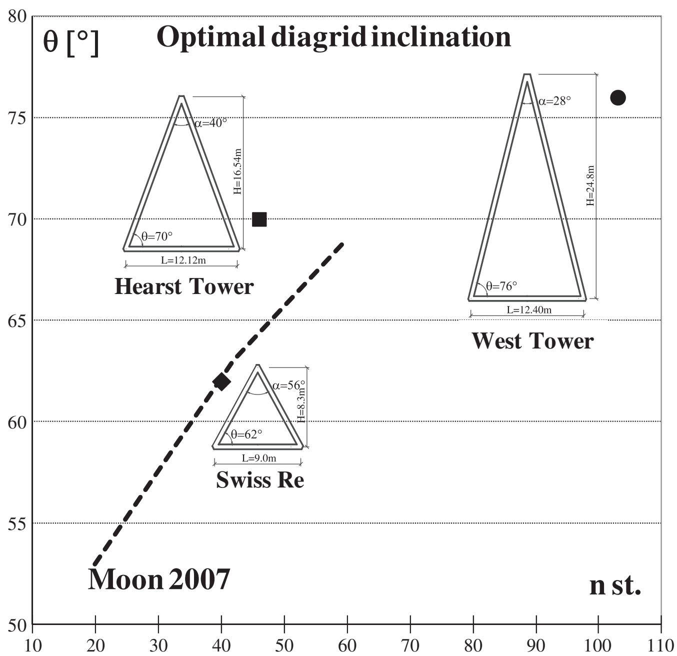

Figure 22. Optimal diagrid inclination—comparison with three case studies.

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- the diagonalized façade gives rise to high horizontal stiffness (total wind sway close to the limit H/500 used in design practice);
- the adoption of optimal values for the unit module geometry as suggested by Moon *et al*. (2007) allows for obtaining an optimized global behavior of the diagrid structural scheme;
- the presence of triangulated units, characterized by elements mainly subjected to axial force, allows for using 'hand calculations' for preliminary design and simplified assessment of diagrid structures.

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