

Ultimate Capacity of Diagrid Systems for Tall Buildings in Nominal Configuration and Damaged State

Giulia Milana, Pierluigi Olmati, Konstantinos Gkoumas, Franco Bontempi

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Abstract

One of the evocative structural design solutions for tall buildings is recently embraced by the diagrid (diagonal grid) structural system. Diagrid, with a perimeter structural configuration characterized by a narrow grid of diagonal members involved both in gravity and in lateral load resistance, requires less structural steel than a conventional steel frame, provides for a more sustainable structure and has emerged as a new design trend for tall-shaped complex structures due to aesthetics and structural performance. The purpose of this study is twofold. First, to assess the optimal structural design of a diagrid tall-building, also compared to a typical outrigger building, focusing on the sustainability (the use of structural steel) and the structural safety and serviceability. To this aim, different diagrid geometries are tested and compared. Second, to provide some insight on the residual strength of diagrid structures, also in the damaged state (modelled by the elimination of diagonal grids). Both goals are accomplished using FEM nonlinear analyses.

Keywords

diagrid · steel-framed · structural design · capacity curves · pushover analysis · structural robustness · sustainability

Giulia Milana

Dipartimento di Ingegneria Strutturale e Geotecnica Sapienza University of Rome, Via Eudossiana 18 - 00184 Roma, Italy
e-mail: giuliamilana@live.it

Pierluigi Olmati

AKT II Consulting Structural and Civil Engineers, 100 St John Street London EC1M 4EH, United Kingdom
e-mail: pierluigi.olmati@akt-uk.com

Konstantinos Gkoumas

Dipartimento di Ingegneria Strutturale e Geotecnica Sapienza University of Rome, Via Eudossiana 18 - 00184 Roma, Italy
e-mail: konstantinos.gkoumas@uniroma1.it

Franco Bontempi

Dipartimento di Ingegneria Strutturale e Geotecnica Sapienza University of Rome, Via Eudossiana 18 - 00184 Roma, Italy
e-mail: franco.bontempi@uniroma1.it

1 Introduction

It is a common understanding that society requires enhanced structures for the people's needs. Besides safety and functional requirements collectively defined in the so-called Performance-based Design [1], today the attention focuses on the sustainability in the broader and profound sense of the word [2].

Sustainability in the urban and built environment is a key issue for the wellbeing of people and society. Sustainable development, defined as the "development that meets the needs of the present without compromising the ability of the future generations to meet their own needs" [3] is nowadays a first concern for public authorities and the private sector. Sustainable design leads to innovation since it demands inventive solutions, and eventually is supported by a cultural shift, evident from national level reviews and surveys [4, 5]. Sustainability issues are wide-ranging in the building industry but the main focus is the reduction of energy consumption in construction and use.

In construction, steel has developed as a material of choice and offers a wide range of solutions that can make buildings more energy efficient, less costly to operate and more comfortable. Several green solutions, aiming at the minimization of structural steel, have been developed in the last few years. Among those, the diagrid structural system is considered as a promising solution for high-rise steel buildings. Diagrid is a perimeter structural configuration characterized by a narrow grid of diagonal members that are involved both in gravity and in lateral load resistance that has emerged as a new design trend recently for tall-shaped complex structures due to aesthetics and structural performance [6, 7].

Since diagrid requires less structural steel than a conventional steel frame, it provides for a more sustainable structure.

This study focuses on the overall structural performance, the ultimate capacity and the robustness of diagrid tall buildings, using numerical (FE) methods. More precisely, the paper is organized in the following manner. Section 2 provides details on diagrid for high-rise buildings. Section 3 outlines structural robustness approaches for steel buildings. Section 4 introduces the case study steel high-rise building and the performed Finite Element (FE) numerical analyses. Finally, Section 5 provides some

considerations and indications for future research.

2 Diagrid structures

Diagrid came as an evolution of the Geodesic Dome invented by Fuller in the late 40's (although as stated in [8], the actual origins are earlier) consisting in triangular structures with diagonal support beams. In fact, the diagrid system is not a new invention. An early example of today's diagrid-like structure is the 13-story IBM Building in Pittsburg of 1963. However, the implementation in a larger scale of such tall building was not practical due to high cost related to the difficult node connections. It is only in recent years that technology allowed a more reasonable cost of diagrid node connections [9]. The Hearst Tower in New York City, is nowadays one of the most iconic and awarded "green" diagrid buildings in the world. Completed in 2006, the 182-meter high building, embraces a highly efficient diagrid frame that uses 20% less steel than a conventionally framed structure. The building is also significant in environmental terms: it was built using 85% recycled steel, and it consumes 25% less energy than an equivalent office building that complies minimally with the respective state and city codes. It is also the first office building in Manhattan to achieve a gold rating under the US Green Building Council's Leadership in Energy and Environmental Design (LEED) programme [10].

In the tall building design of diagrid and braced tube structures for example, stiffness provision is an important aspect of the design, and a material saving design methodology can be based on this aspect [11]. In fact, diagrid structures can be seen as the latest mutation of tube structures, which, starting from the frame tube configuration, have increased structural efficiency thanks to the introduction of exterior mega-diagonals in the braced tube solution.

The perimeter configuration still holds 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.

Tall buildings in general need to respect certain safety and serviceability thresholds (e.g. the inter-story drift and acceleration), that are related to the damage to the partitions and claddings and the occupant comfort (see for example [12]). Furthermore, they have to be sufficiently robust, to withstand exceptional actions and abnormal situations [13]. These aspects need to be assessed for innovative structural systems such as the diagrid configuration.

A diagrid structure is modeled as a vertical cantilever beam on the ground, and subdivided longitudinally into modules according to the repetitive diagrid pattern. Each module is defined by a single level of diagrids that extend over multiple stories.

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 [6]. 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 [14], 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 [7].

The optimal structural configuration of diagrid structures is nevertheless trivial. Recent research focuses on the optimal layout of diagrid structures, which is very complex and carried out with optimization methods [15]-[16] and the alternative geometrical configurations, obtained by changing the angle of diagonals as well as by changing the number of diagonal along the building height [17]. The same authors [18] propose secondary bracing systems for diagrid structures, capable of improving stability and local flexibility, at the cost of a small increase in structural weight.

3 Steel buildings and robustness

The residual capacity in the damaged state and the structural robustness of steel buildings has been thoroughly studied in the last years. Even though a variety of terms have been used in literature, robustness in structural engineering is commonly defined as the "insensitivity of a structure to initial damage" and is correlated to collapse resistance, intended as the "insensitivity of a structure to abnormal events" [19].

A review of international research on structural robustness and disproportionate collapse is provided in [20]. In [21] an overview of literature on the topic of robustness, progressive collapse, and disproportionate collapse is provided. In [22] design strategies are identified for obtaining robustness, using prevention and mitigation measures. The author concludes that one of the widely used methods to assess the structural robustness of a structure, is to consider the removal of a key element, and check the vulnerability of the structure. The removal of key elements in a scenario-based approach is actually a common and direct method for checking the robustness of a structure.

In [23] focus is given on the vertical load bearing capacity of truss structures, using a sensitivity index that accounts for the influence of a lost element to the load bearing capacity. In [24] nonlinear dynamic analysis are conducted on benchmark buildings (3, 6 and 15-story) and the results are compared with more straightforward linear static step-by-step analysis. In [25] a framework is provided for the progressive collapse assessment of multi-story buildings, considering as a design scenario the sudden loss of a column. Using this framework, the same authors [26], investigate possible scenarios, in the form of the removal of either a peripheral or a corner column, in a typical

steel-framed composite building. In [27] different column loss scenarios on 3 and 4-story steel buildings are investigated, focusing on different aspects of the problem, among else, the load redistribution.

In [28] focus is given on the progressive collapse of both traditional (tubular) and diagrid structures, conducting nonlinear static and dynamic analyses, considering scenarios of column loss (different corner columns for the tubular structure and pairs of diagrids). More recently, in [29] the progressive collapse of diagrid structures is assessed using nonlinear static pushdown analyses, considering the removal of structural elements, and by gradually increasing the vertical displacement in the location of the removed elements.

In this study, as a preliminary step, an event based design approach is implemented, using pushover analysis (see for example, [30]) on diagrid structures subjected to scenarios concerning the removal of different columns or pairs of columns. As in the case of the undamaged state, the residual strength (pushover) analysis is carried out under horizontal loads having a triangular distribution along the height of the building. This choice is made with two motivations in mind [31]: i) horizontal loads can activate both horizontal and vertical load bearing structural systems of the building and ii) direct reference is made to the unlikely eventuality that a seismic aftershock occurs after an explosion. This event is possible in the case that the explosion occurs after a seismic main shock (e.g. hydrogen explosions caused by the Japan 2011 earthquake main shock).

4 Numerical modelling

The considered structure is a 40-story building, for a total height of 160 m, and a footprint of about 36 m x 36 m. Its function is for non-public offices (carrying anthropic loads of 2 kN/m²). The building is located in Latina (Lazio, Italy). Regarding local wind and earthquake loading conditions (in compliance with the Italian Building Code [33]), the area where the building is placed is characterized by a class of roughness “B” (urban and sub-urban areas) and a class of exposition to wind “IV” (with a wind reference average 10 minute speed of 27 m/s). The seismic zone corresponds to a class II seismic level (PGA 0,15 - 0,25).

In the preliminary design, comparisons are performed to select the most efficient structural system and to reduce the material used, between two proposed different structural design solutions: outrigger and diagrid. Typically, a diagrid structure is subdivided longitudinally into modules according to the repeated diagrid pattern. Each module is defined by a single level of diagrid that extends over multiple stories. In the building here presented, there are 4-story modules.

Some generic considerations are recalled. The optimal angle of diagonals is highly dependent upon the building height. Since the optimal angle of the columns for maximum bending rigidity is 90 degrees and that of the diagonals for maximum shear rigidity is about 35 degrees, it is expected that the optimal angle of

diagonal members for diagrid structures will fall between these angles. Fig. 1 shows the structural systems of the considered buildings, together with the reference outrigger structure.

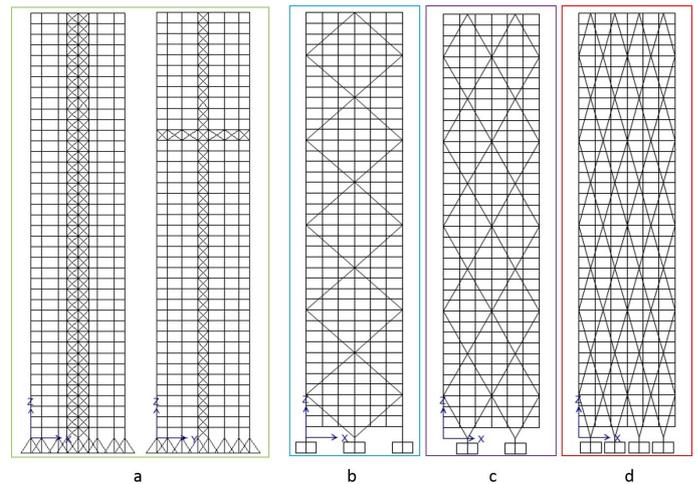


Fig. 1. Structural Configurations (a- Outrigger Structure: ZX and ZY plane at respectively $Y = 9,5$ m and $X = 4$ m; b- Diagrid Structure $\alpha = 42^\circ$; c- Diagrid Structure $\alpha = 60^\circ$; d- Diagrid Structure $\alpha = 75^\circ$)

As can be seen, this study introduces three intermediate angles: 42, 60 and 75 degrees respectively. The reference outrigger structure (for additional modelling details see [32]) has a plan symmetric with respect to the X-axis, and an octagonal footprint, approximated by a square of 35 m x 35 m. The distance between two consecutive floors is 4 m and the structure has been realized in order to make a diagonal bracings system resists horizontal actions of the wind. The diagonal elements of the system consist in St. Andrew cross-bracings. In order to reduce the building deformations, a rigid plane is introduced, identified as “outrigger”. This “reinforcement”, located at the 29th floor (between 112 m and 116 m), is realized by introducing braces expanded vertically for all façades in exam. These outriggers are located on two façades at $Y = \{4, 31\}$ (centered) and on the other two façades at $X = \{4, 31\}$ m

The three diagrid buildings have two structural systems working in parallel: the first is internal and it is made of a “rigid” frame system which only reacts to gravity loads, while the second is perimetral and it is made of a diagonal grid system which reacts both to vertical and horizontal loads. The reader is referred to [6] for modelling details. The internal structure, as any other ordinary frame structure, is composed by columns and main and secondary beams, while, the external one is composed by diagonal and horizontal elements (without columns).

All the components of the internal system are placed at a distance of 6 m in plan, thus creating square footprints of 6m x 6m. The internal columns transmit vertical loads to the ground, while the perimetral ones do not; in fact their function is to link the generic diagrid module to the floors included in it. In more details, the external columns receive the loads from the perimetral beams and transfer these loads to the horizontal elements of the module. The extension of the external columns is four-story

length as the diagrid module. Passing from one module to the consecutive one, the perimetral beams are replaced by the horizontal diagrids. In this way, the two structures “communicate” every four floors. All of the vertical elements are tapered every four stories, since the size of each diagrid module changes. Appropriate steel profiles are used for the interior structure and for the perimeter structure.

For all structures, the floors have been modeled using shell elements that reflect the real resistance and stiffness of the floors. Foundations have not been explicitly modelled, and pinned joints have been used in order to model connection between the structure and the ground. Since the principal objective of this study is to compare different structural systems, this approximation can be considered as reasonable.

The computational code SAP2000 (version 16.0.0) is used for the analysis. The structural model takes into account the real distribution of the masses, while the effect of non-structural elements on the global stiffness has not been considered.

In this study the weight reduction is the most important issue, since this is considered as the most significant aspect from a sustainability point of view. For all diagrid buildings an important weight reduction occurs, therefore, in all cases the diagrid system results better than the ordinary outrigger for what regards sustainability.

The weight of the structures is calculated without considering the floors; this is a simplification arising from the fact that all the structures have the same number of floors.

Table 1 presents the comparison among the structures in the preliminary design. The percentage of reduction is calculated compared to the weight of the outrigger structure.

Tab. 1. Weight and weight reduction

Structure	Weight (ton)	Weight-reduction (%)
Outrigger	8052	-
Diagrid 42°	6523	19
Diagrid 60°	5931	26
Diagrid 75°	5389	33

4.1 Structural checks

It is important to verify the structural configurations for both Serviceability Limit States (SLS) and Ultimate Limit States (ULS). To this aim, displacements are compared with thresholds provided in the Italian National Code, and pushover analyses are performed.

For a first check of the serviceability limit states, the total horizontal displacements are considered. The threshold SLS value provided by the Italian Building Code [33], is set at 1/500 of the total height of the building (corresponding to 0.32 m for a height of 160 m). Additional checks are performed for the inter-story drift, nevertheless, Fig. 2 reports the total horizontal displacements both at top and those corresponding to points of control placed every four stories (16 m or one diagrid module), at the

corresponding height since the check is more restrictive compared to the inter-story drift specified in [33].

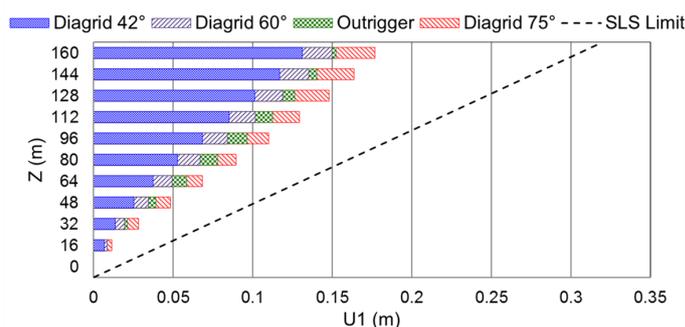


Fig. 2. Comparison of horizontal displacements

It is straightforward that all four structures are verified by a great margin, with the Diagrid 42° being the stiffest, the Diagrid 75° the most flexible, and the remaining two having an intermediate and very similar performance.

For the verification of the ultimate limit states, and in order to evaluate the ductility of the structures, a non-linear static (pushover) analysis is conducted. A lamped plasticity model has been implemented taking into account the material non-linearity. For simulating this non-linearity, plastic hinges are used. The Pushover analysis is conducted on the 3D model for all structures with the same static loads and hinges, in order to have a direct comparison of the results.

The horizontal load applied to the structure is a triangular load, increasing with height. The concentrated forces, are applied to the geometric centers of each floor and represent the equivalent normalized static forces.

Two different kinds of hinges are considered: axial hinges, used for all elements of the outrigger structures and the perimetral system in the diagrid structures, and, bending hinges for the internal columns in the diagrid structures. For the axial hinges, the constitutive equation is rigid and perfectly plastic, with a yield stress equal to f_{yk} (430 MPa for diagrid structures and 275 MPa for the outrigger structure) and a fracture deformation (ϵ_u) of 5%. For the bending hinges, the behavior is defined through a moment-curvature diagram with a rigid and hardening-plastic constitutive equation.

For the bending hinges the behavior is defined through a moment-curvature diagram with a rigid and hardening-plastic constitutive equation. The final curvature is extrapolated using Table 5-Table 6 of FEMA356 [34] while the other parameters are calculated in this way:

$$M_y = f_{yd} \cdot W_{el} \text{ and } M_u = f_{yd} \cdot W_{pl} \quad (1)$$

$$\chi_y = \epsilon_y / (h/2) \quad (2)$$

where, M_y is the yield moment, M_u is the final moment, W_{el} is the elastic modulus, W_{pl} is the plastic modulus, χ_y is the yield curvature and ϵ_y the yield strain.

The hinge length is defined as being equal to the height of the cross section. These hinges are placed in the interior columns (in the diagrid structures) at the top and the bottom of each story.

Figs. 3-5 report the comparison of the capacity curves of all structures. The horizontal displacements (X-axis) are measured at the top of the structure (at a height of 160 m). In order to simplify the reading of results all graphs are presented with the same scale axis.

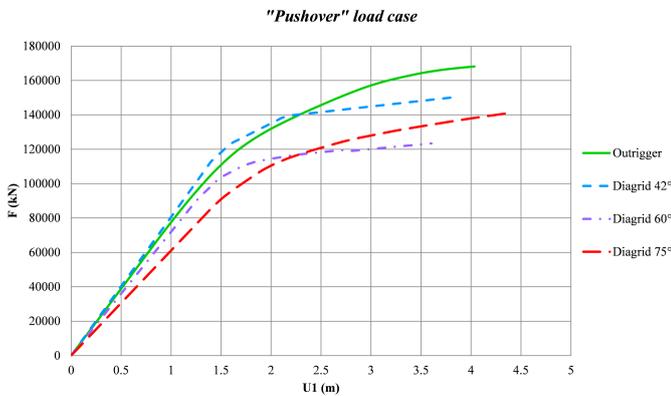


Fig. 3. Comparison of capacity curves for the “Pushover” load case

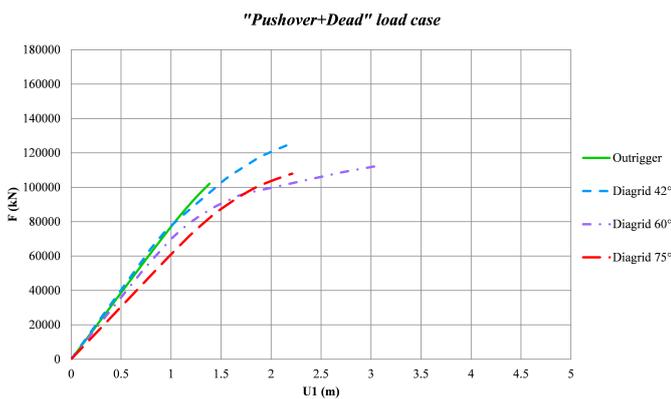


Fig. 4. Comparison of capacity curves for the “Pushover + Dead” load case

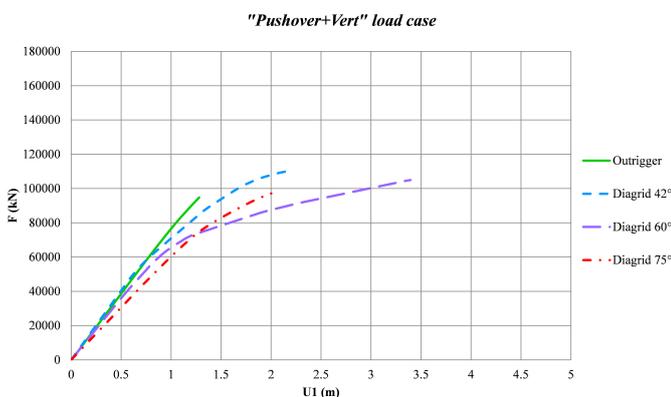


Fig. 5. Comparison of capacity curves for the “Pushover + Vert” load case

The “Pushover” load case is purely theoretical, since only the horizontal loads are considered. In order to consider the effect of geometric non-linearity in the structural behavior, a P-Delta non-linear static analysis is introduced. The P-Delta ef-

fect refers specifically to the non-linear geometric effect of a large tensile or compressive, direct stress upon transverse bending and shear behavior. In order to take into account the effect of gravity loads upon the lateral stiffness of building structures, a non-linear case is created considering only the dead loads of the structure (“Pushover + Dead” load case) and one that considers all vertical loads, including dead, permanent and cladding loads, and excluding live loads (“Pushover + Vert” load case).

The ideal “Pushover” load case (Fig. 3), provides the most marked results. This purely theoretical load case (absence of vertical loads), allows the model to reach a high number of steps. The Diagrid 42° and the Outrigger structures present a clear advantage compared to the other two structures. For horizontal displacements limited to 2 meters, the performance of the Diagrid 60° is also superior to the Diagrid 75°. The same trend is repeated for the other two load cases (Figs. 4 and 5), with the P-Delta effects, although in a less marked but more realistic manner. In these cases, the analyses are interrupted earlier, probably due to non-convergence of the solution. Fig. 6, shows the plastic hinge formation for the “Pushover” load case, for the Diagrid 42°, Diagrid 60° and Diagrid 75° 3D models. The analyses stop at Step 65, 67 and 72 respectively.

4.2 Comparison and choice of the best model

Based on the capacity curves of section 4.1, it is possible to obtain three of the four values from which we can identify the model with the best behavior. These properties are:

- R Strength
- K Stiffness
- μ Ductility

For the analyses, the same considerations made in the previous section remain valid.

One considers Fig. 7 for calculating these properties; the capacity curve in this figure is an ideal example of the realization of these aspects that are:

- D_y yield displacement
- D_u maximum displacement
- F_y yield force
- F_{max} maximum force

From these aspects, it is possible to obtain the mechanical properties of interest in the following way:

$$R = F_{max} \text{ (Strength)} \quad (3)$$

$$K = F_y / D_y \text{ (Stiffness)} \quad (4)$$

$$\mu = D_u / D_y \text{ (Ductility)} \quad (5)$$

Using these properties as well as the weight of the structure, the buildings are compared and the best structure is chosen through a performance index defined in the following paragraph.

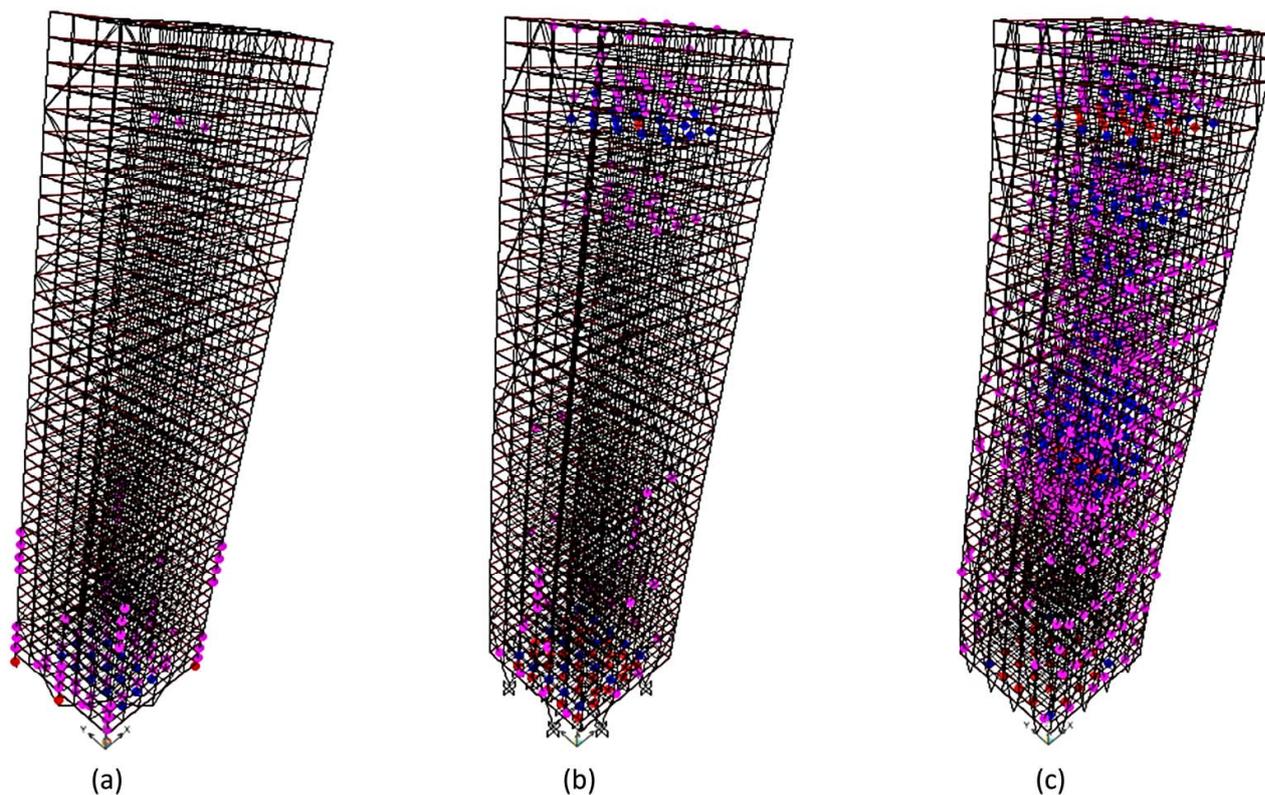


Fig. 6. Hinge formation for the “Pushover” load case (a- Diagrid Structure $\alpha = 42^\circ$; b- Diagrid Structure $\alpha = 60^\circ$; c- Diagrid Structure $\alpha = 75^\circ$)

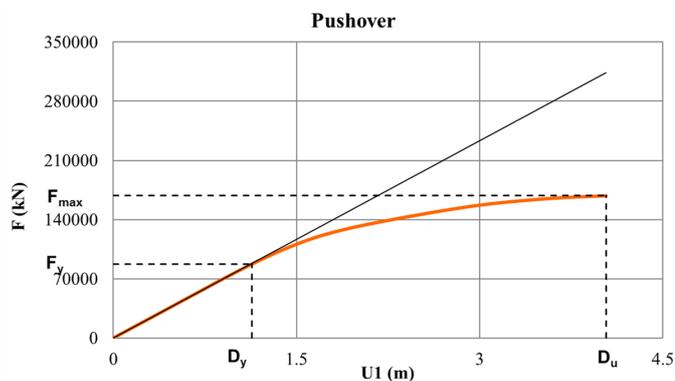


Fig. 7. Definition of the main aspects of the capacity curve

All these aspects are calculated just for the “Pushover + Vert” load case, since it is the most realistic case.

In order to assess the performance of the structures, in a first place, the following performance factors are used:

- 1 the stiffness-to-weight (K/P) ratio (specific strength) for the SLS;
- 2 the strength-to-weight (R/P) ratio (structural efficiency) for the ULS.

These are calculated for each structure in the following manner, normalized to the aspects of the outrigger structure, that is, the reference building:

$$SLS: (R/P)/(R_0/P_0) \quad (6)$$

$$ULS: (K/P)/(K_0/P_0) \quad (7)$$

The subscript “0” identifies the aspects relative to the outrigger structure. Table 2 reports the results.

Tab. 2. Comparison of models for the ‘Pushover + Vert’ case

	OUTRIGGER	DIAGRID 42°	DIAGRID 60°	DIAGRID 75°
ULS	1	1,435	1,504	1,531
SLS	1	1,290	1,255	1,179

As can be seen from the results, the diagrid 60° structure has an intermediate performance compared to the other diagrid structures, while the diagrid 42° gives the best results in the SLS, and the diagrid 75° the best in the ULS.

At this point, in order to obtain a better view of the global behavior of each structure, an additional performance index is

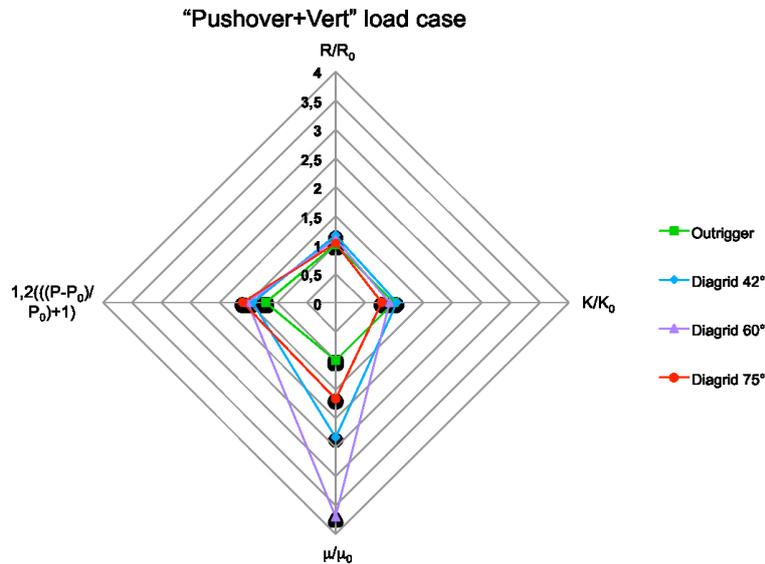


Fig. 8. Comparison of models for the “Pushover + Vert” case

introduced in Eq. 8, that helps to identify the structure with the best behavior.

$$(R/R_0) + (K/K_0) + (\mu/\mu_0) + 1,2(((P_0 - P)/P_0) + 1) \quad (8)$$

Again, all terms of this index are normalized to the aspects of the outrigger structure. These terms are multiplied with amplification coefficients. For the weight ($P \leq P_0$), a coefficient equal to 1.2 is considered, while for the other terms the coefficients are equal to 1. The higher coefficient is used since weight is very important for the sustainability aspect, and the weight reduction is encouraged in the global assessment. The higher the outcome, the better the behavior of the structure.

Table 3 provides the results of Eq. (8) for each structure. The terms of the equation, multiplied for the relative coefficients, are shown on the axes of a radar chart (Fig. 8).

As can be observed from Table 2 and Fig. 8, the overall best configuration is provided by the “Diagrid 60°” configuration, with diagonal members having an inclination of 60°. Thus, the diagrid structure with an intermediate inclination results as the best model.

In fact, this structure leads to an important reduction (although not the largest) of weight while at the same time, offers a high performance in terms of strength, stiffness and ductility.

Ultimate capacity of the optimal diagrid configuration

Different analyses are carried out on the optimal diagrid configuration (Diagrid 60°). First, the pushover curve of the “Pushover” load case of Fig. 3 is expanded for the specific model. Thus, in Fig. 9, the plastic hinge formation is also shown. Similar results are obtained for the other two pushover load cases (“Pushover + Dead” and “Pushover + Vert”). Even though the latter cases are more realistic, it was chosen to show only the former for the sake of brevity, since also the capacity curves are more marked in the absence of P-Delta effects.

The perimetral frame has essentially a regular behavior, with hinges forming on the base of the lower module (Step 25) and propagating gradually to the upper modules. The behavior is globally good, since the hinge formation occurs for very large displacements. More specifically, regarding the hinge formation at the base of the structure, some important aspects are:

- at step 25, the hinge formation initiates on the diagonals of the lower diagrid module;
- at step 37, hinges are formed in the internal columns at the base;
- at step 44, the hinges at the base reach the immediate occupancy limit, identified by SAP2000;
- at step 51, the hinges at the base reach the ultimate capacity limit, and consequently, the structure collapses;
- at step 67, even though hypothetical since the analyses are based on a displacement control, hinges are formed at the 1st floor of the internal columns.

4.3 Ultimate capacity in the damaged state

Additional analyses are carried out for damaged configurations of the Diagrid 60° structure. In particular, six different damage scenarios, accounting for the elimination of one or two adjacent diagonals along the building height, are considered. With reference to Fig. 10, “Di,Lj” indicate the (i-th) damage scenario and the elimination of a number of (j) elements (e.g. D1,L2 corresponds to the elimination of two diagonals for scenario 1).

The scenarios depict the possible damage from an exceptional event (e.g. explosion), and are modeled by introducing zero resistance elements in the place of the diagonals.

In order to evaluate the capacity curves of the structure in the damaged state, two of the load cases introduced in Section 4.1 are considered: “Pushover” and “Pushover + Vert”.

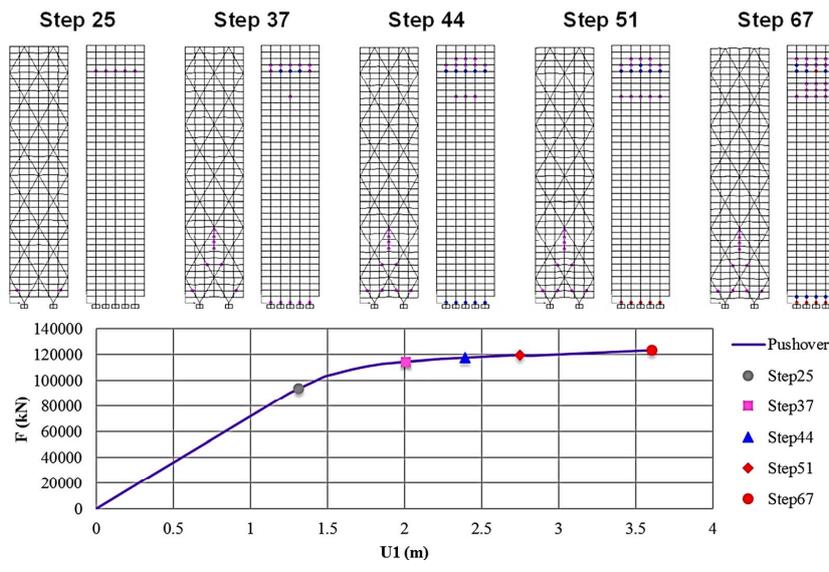


Fig. 9. Capacity curves on the "Diagrid 60°" model for the "Pushover" load case

Tab. 3. Comparison of models for the 'Pushover + Vert' case

		OUTRIGGER	DIAGRID 42°	DIAGRID 60°	DIAGRID 75°
R	(KN)	94775	110185	104972	97131
K	(KN/m)	77143	80615	71306	60897
μ		1,535	3,587	5,681	2,564
P	(ton)	8052	6523	5931	5389
Performance index (Eq. (8))		4,20	5,97	7,25	5,08

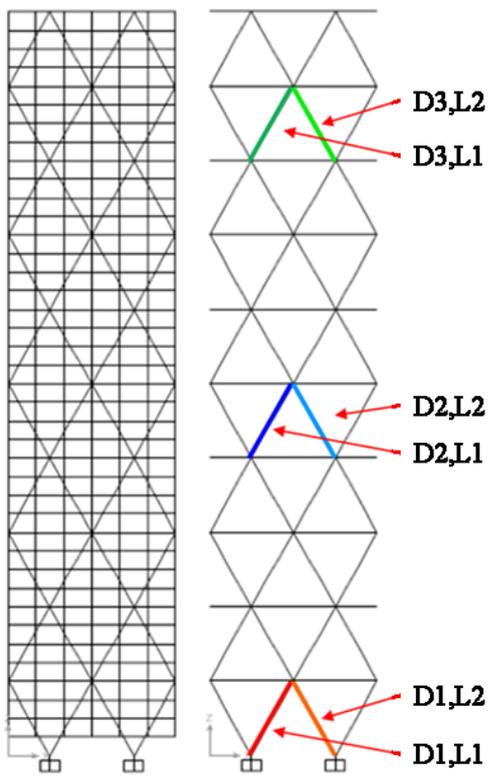


Fig. 10. Damage scenarios for the "Diagrid 60°" structure

For both cases, it was necessary to deal with the problem of interrupting at some point the capacity curves. Thus, two cases are identified:

- i. a case where the capacity curves are interrupted at the reach of the ultimate capacity of a single (or more, if occurring simultaneously) plastic hinge (this case is identified as "First Plastic Hinges")
- ii. a case where the capacity curves are interrupted at the reach of the ultimate capacity of the highest number of plastic hinges (this case is identified as "Last Plastic Hinges")

In most cases, the hinges that reach the ultimate capacity are those corresponding to the base of the internal columns. However, in some cases, principally for the analyses with P-Delta effects, it was not possible to identify with precision the point in which the curve is interrupted, since the analyses are interrupted due to non-convergence of the solution.

For the case in which the damage state corresponds to the vertical plane, Table 4 provides the steps in which the capacity curves are interrupted, together with the number of internal columns for which the hinges on their base reach the ultimate resistance.

It should be noted that the total number of internal columns is 25. The behavior in the "Pushover + Vert" (P + V) load case (which account for the P-Delta effects) is of difficult interpretation, since the analyses are interrupted for non-convergence

Tab. 4. Comparison of models for the ‘Pushover + Vert’ case

Damage Scenario	Analysis	First plastic hinges		Last plastic hinges	
		STEP	Base plastic hinges	STEP	Base plastic hinges
D1,L1	P	42	3	48	25
	P + V	31	4	32	5
	P + V lin	33	4	44	15
D1,L2	P	49	25	49	25
	P + V	54	12	55	23
	P + V lin	55	25	55	25
D2,L1	P	50	25	50	25
	P + V	35	-	35	-
	P + V lin	56	12	57	25
D2,L2	P	51	17	52	25
	P + V	39	-	39	-
	P + V lin	54	25	54	25
D3,L1	P	51	25	51	25
	P + V	40	-	40	-
	P + V lin	51	25	51	25
D3,L2	P	51	25	51	25
	P + V	38	-	38	-
	P + V lin	54	15	55	25

of the solution, rather for reaching the ultimate capacity of the hinges. For this reason, an additional pushover analysis is considered, identified as “Pushover + Vert lin.” (P + V lin). This load case is the same as “Pushover + Vert” (thus, considers all vertical loads), but without accounting for the geometric non-linearity (the P-Delta effects).

It is possible to observe that, mainly for the “Pushover” case, the realization of the ultimate capacity of the hinges on the base of the columns is simultaneous, thus the two cases of first and final plastic hinges coincide.

Comparatively, and considering for the sake of brevity the “Pushover” load case, as Fig. 11 suggests, the capacity curve drops significantly for the case D1,L2, while the case D1,L1 gives an intermediate response compared to the nominal configuration.

Nevertheless, the residual capacity is generally good, considering the importance of the elements, since the performance drop is approximately 20% and 10% respectively. In the D2,L1 and D2,L2 scenarios, the capacity drop is very small (less than 2%), while in the D3,L1 and D3,L2 scenarios, the capacity drop is negligible. Therefore, also the curves of Fig. 11 for these cases are practically superimposed.

5 Conclusions and considerations

The inspiration for this study arises from the impact that the construction industry has on the environment, in terms of use of resources and production of waste, and the social need that calls for investigating sustainable solutions, such as the considered sustainable diagrid high-rise buildings.

Among the finding, the way in which diagrid structures lead to a considerable reduction of (steel) material compared to more

traditional structural systems such as outrigger structures is quantified. Furthermore, the performance of diagrid structures has been assessed, not only in terms of structural steel reduction, but also in terms of safety, serviceability and structural robustness.

Different diagrid structures were considered, namely, three geometric configurations with inclination of diagonal members of 42°, 60° and 75°. These configurations, in addition to allowing a considerable reduction of weight, provide a better performance in terms of strength, stiffness and ductility.

Between the considered diagrid structures the one with the best overall behavior results to be the one with 60° diagonal element inclination. For this configuration, additional analyses in the damage state are performed, assessing the overall good performance of the structure under exceptional events.

Nevertheless, there are some limitations to this study. Additional loading scenarios should be accounted for, in order to have a broader insight on the structural behavior. In addition, the defined performance index is calibrated with specific coefficient values that highlight the sustainability aspect. Finally, robustness checks should be applied using appropriately defined indexes. Future research could also concentrate on the modelling of local buckling, using appropriate software and FEM models.

In any case, the initial results provide a starting point, and together with the proposed methodology, contribute obtaining a preliminary assessment of the sustainability and structural performance of tall diagrid structures.

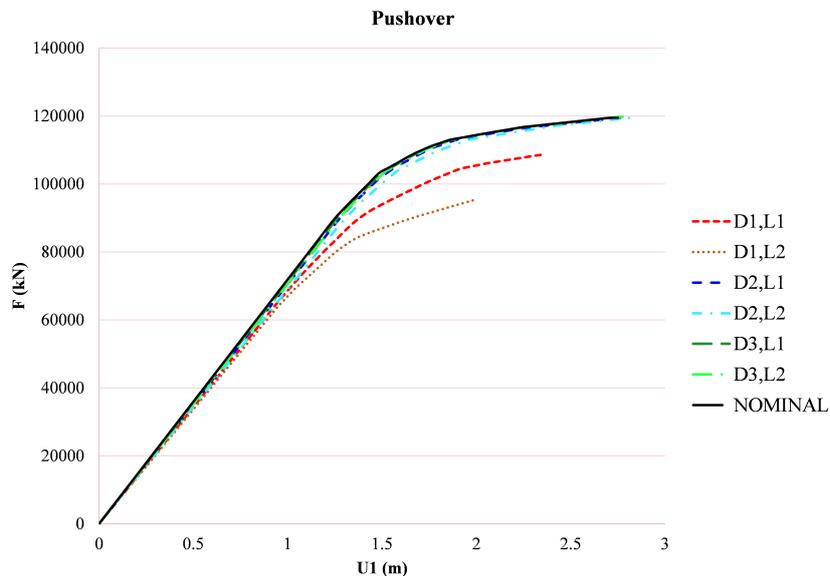


Fig. 11. Capacity curves for the “Diagrid 60°” model for the “Pushover” load case and for the different damage scenario

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