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Article

Geometrical Issues on the Structural Analysis of Transmission Electricity Towers Thanks to Laser Scanning Technology and Finite Element Method

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Abstract: This paper presents a multidisciplinary approach to reverse engineering and structural analysis of electricity transmission tower structures through the combination of laser scanning systems and finite element methodology. The use of laser scanning technology allows the development of both drawings and highly accurate three-dimensional geometric models that reliably reproduce geometric reality of towers structures, detecting imperfections, and particularities of their assembly. Due to this, it is possible to analyze and quantify the effect of these imperfections in their structural behavior, taking into account the actual geometry obtained, different structural models, and load hypotheses proposed. The method has been applied in three different types of metal electricity transmission towers with high voltage lines located in Guadalajara (Spain) in order to analyze its structural viability to accommodate future increased loads with respect that which are currently subjected.

Keywords: non-destructive techniques; structural analysis; Laser Scanning; Finite Element Method; transmission towers; reverse engineering

1. Introduction

Traditionally, building high-voltage power lines has had few obstacles during their construction phase. Currently, this type of infrastructure is facing a number of setbacks: it has a considerable impact on the environment, on economic activities, and on the expansion of cities, besides its economic cost, including inspections and maintenance. All of these problems have led the companies that use and maintain this infrastructure to consider making the best use possible of the existing lines before placing new lines. Old lines were designed according to the standards of the time in which they were built and they were designed to bear a certain load. In many cases, these towers were designed over forty years ago, so the increased loads that will be placed on them will be far greater than the one for which they were designed. In addition to this fact, the design and execution data of the towers has, in most cases, disappeared, and in other cases, building regulations did not even exist at the time. Due to this, addressing the re-use of existing power lines requires a geometric and structural analysis of the towers to assess their current state.

Formerly, the towers' dimensional analysis was performed through expeditious and manual methods (through the use of a gauge and a measuring tape) that required direct contact with the structure and, therefore, meant high risks and high costs. Afterwards, in search of a remote non-invasive measuring method, classic topographic measuring allowed thorough, notably intense, field work taking angular measurements and determining singular points indirectly through angular intersections. More recently, the existence of reflectorless electromagnetic surveying equipment has allowed direct measurement of distances and angles from a single point, making field work easier and more efficient, although it solely focuses on extracting unique and specific measurements determined by the topographer [1]. This has meant great uncertainty upon the elements of the tower, since the data was only taken at the point where the measurement is performed. In order to have the full representation of the geometry of the structure, in the last years laser scanning has presented as an interesting solution [2,3,4,5], due to the fact that they generate dense real-time point-clouds of the tower's geometry from a distance [6]. However, one of the major limitations of these terrestrial geotechnologies is the overall height of the tower, impossible to cover completely from the ground, which has led to the use of robotic unmanned aerial systems that take aerial images and, through photogrammetric procedures, obtain dense three-dimensional models of this type of infrastructure [7].

As for structural assessment, this kind of structure has been analyzed from different points of view as presented in the literature: the effects of loading in the stability of the tower [8–11]; the effects of the stiffness of connections [12–14]; and causes of failure [15–17]. No previous work was performed in order to evaluate the effect of geometric imperfections such as misalignments of structural members at joints or assembly imperfections.

Therefore, in view of what is mentioned above, this paper presents a non-destructive multidisciplinary approach that is articulated in two stages and that analyzes geometrically and structurally electricity transmission tower structures:

1. The first stage will address a detailed geometric description of the structure (reverse engineering) using a terrestrial laser scanner system, performing an "as built" model that provides information on the structure's most relevant data, such as geometry, type, and dimensions of the metallic profiles and their assembly drawings.
2. Secondly, and taking the geometric model obtained by the laser system as a starting point, three different structural finite element models will be developed: one model will have an ideal geometry considering the nodes of the transmission tower to be pinned, supposing that this model was the one that was used in the original design of the towers. A second model will be developed with the ideal geometry of the previous model, but considering certain continuous elements of the transmission tower to be rigidly connected at both ends. The third model will use the real geometry obtained from the laser scanner (taking into account all possible initial imperfections as, for example, misaligned structural members at joints) and will also consider the stiffness behavior of the continuous elements of the tower. For its analysis the current Spanish standard for the design of such towers will be taken into account [18]. The analysis will be carried out in a linear elastic regime with the software SAP2000 [19] to obtain data of the displacements and stresses in transmission tower members for each load case established by the current codes. It is expected to obtain conclusions about the performance of these different structural approaches and, therefore, conclude which is the most appropriate modeling strategy in structural assessment procedures when an increase in loads to accommodate new services is demanded.

The paper is outlined as follows: after the introduction, in Section 2, the characteristics of the towers' considered structural models are described. Section 3 details: the equipment used, the production of the geometric models through computer aided design (CAD) software, and the data used in structural analysis. In section 4 the results are analyzed and end with the reached conclusions.

2. Structural Modeling of Truss Structures

A transmission tower could be considered as a three dimensional truss structure [20], which is comprised of a reticular structure formed by discrete elements (bar or rods), joined together at their ends by means of connections without friction and designed to withstand the external forces by means of axial internal forces in their members. The idealized structural model used for the study of this kind of structure is usually based on the following assumptions [21,22]:

1. Individual elements or rods are joined together at their ends by means of connections with no friction, which means the nodes transmit forces but do not transmit moments.
2. The centroidal axis of each member is straight and matches the line that joins each end of the member. The axis of all of the members that end in the same node is cut at a single point; otherwise moments will appear in these members so that the node could be at static equilibrium.

3. Whenever external loads are applied in the nodes, all the elements that comprise the structure are subjected to tensile and compressive forces, since there is no friction at the connections. This means that the self-weight of the elements is replaced with loads applied at their ends. According to [23] bending caused by direct wind loads on the structural elements can be omitted.
4. The cross section of each member has a negligible area compared to its length.
5. The self-weight of the elements is negligible, since the loads supported by the structural elements are usually large in comparison to their weights.

Under the fulfillment of these assumptions, the structural elements are exclusively subjected to axial forces. For the case studies later reported in this paper, given the age of the towers and the virtual inexistence of structural calculation software at that time, it is logical to assume that they were either calculated manually, using the so-called Ritter method [21,22] or graphically, using the Cremona method [21,22].

However when looking at the analyzed tower's "as built" model we found out that assumption 1 and 2 are not true in the case studies herein presented:

1. In the case of members of the towers, their connections are bolted and thus they are not actually frictionless nodes. Additionally, some members, such as the truss chords, are continuous elements, which can evidently transmit moments from one side of a node to another [24].
   Thus, these nodes are assumed to be rigidly connected behaving as elastic embedment (or simply rigid joints), which cause local bending moments due to rotations that take place as a consequence of the global deformation of the structure. Such effect causes the so-called secondary bending moments and consequently a secondary stress state, which are juxtaposed to the primary stresses derived from axial forces.
2. Furthermore, truss members that end in the same node do not always cut it at the same point, leading again to an apparition of secondary bending moments. Such bending moments depend upon both proportion of the misaligned or eccentricity and stiffness of the elements.

Both circumstances are observed in all towers herein analyzed. Figure 1 corresponds to an example of the aforementioned issues upon one of the case studies later described (Tower 1). Furthermore, as it can be appreciated, the ideal situation of perfectly-aligned elements assumed in the initial stage of structure design and calculation can suffer variations during its construction on site. This is shown in the "as built" model, obtained with the laser scanner, where small deviations in the horizontality of several truss members and even a small deviation in the vertical alignment of the main body of the tower can be appreciated (Figure 1).

All of these issues question the use of a model with ideal pinned connections, which was possibly the used method at the time of the structure's design and calculation, and whether it is a valid model to perform further calculations at the present time. Therefore, in view of the constraints and considerations discussed above, three different structural models of the towers are tested in order to compare the results with each other and draw conclusions about the deformations and the actual stress state of the tower's elements, all under different load hypotheses established by current standards [18].

1 Model with pinned joints and ideal geometry (Model 1). This model is supposed to be the one that was used for the design of the towers. A three-dimensional model with all members

pin-jointed at nodes was developed that took into account all the previously-mentioned assumptions. Accordingly, the adoption of certain assumptions about the geometry provided by the laser scanner was needed in order to adapt the model to an ideal geometry. More specifically, it was assumed that all members concurring in one node were overlapping in a single common point since, in reality, these members were assembled more erratically. For this, the criterion used was to assume that the main vertical members of the tower and the horizontal members were fixed in their position, and the corrections changed the length and exact location of the diagonal braces that connect at each joint. Additionally, the geometry was simplified in certain areas of the horizontal sections of the towers, ignoring the bend and the inclination. Accordingly, vertical deviation of the main bodies of the towers was not taken into account. Finally, the obtained "as built" model measurements of certain modules that form the main body of the towers are simplified to theoretical values. For example, when certain lengths result in 1.99 m or 2.01 m, the value is simplified and considered to be 2 m. For clarity, Figure 2 presents a scheme with the principal structural parts of a transmission tower.

2 Model with rigid joints and ideal geometry (Model 2). According to the data proportioned from the "as built" model, assumption 1 is not valid because certain elements, such as truss chords, are continuous elements. Therefore, a second model was elaborated which considers an ideal geometry but where truss chords and horizontal members of the towers are considered rigidly connected at both ends and all other structural components (truss elements) are treated as pin jointed. In this way, the originated secondary bending moments and, consequently, secondary stresses can be taken into account and their impact in comparison to modeling the tower with all ideal pinned joints quantified.

3 Model with rigid joints and real geometry (Model 3). In contrast with the two previous models, this approach considers all of the real geometric singularities of the structure and includes them into the simulations; for example, it includes horizontal deviations of truss chords, the vertical deviation of the tower's main body, as well as assumptions 1–3 obsolescence. To create such a structural model, first a 3D wire-model was produced using the "as-built" model as a dimensional foundation. Secondly, it was imported as a "dxf" file into the structure calculation software SAP2000 [19].

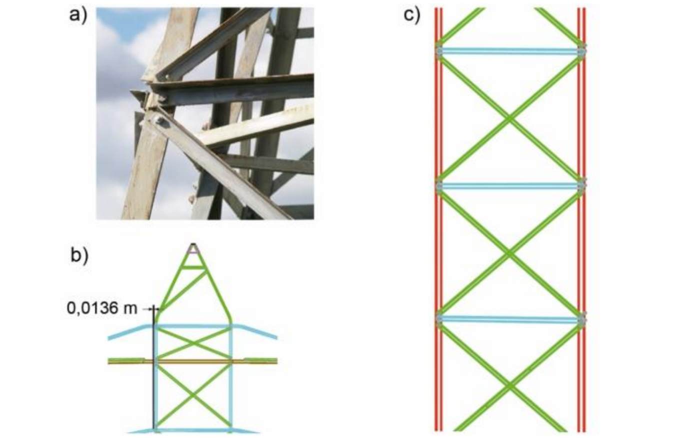

Figure 1. (**a**) Detail of the bolted connections with the inclusion of the eccentricity present at the linkage between profiles; (**b**) vertical deviation of the main body of tower; (**c**) horizontal deviation in the members of the main body of tower.

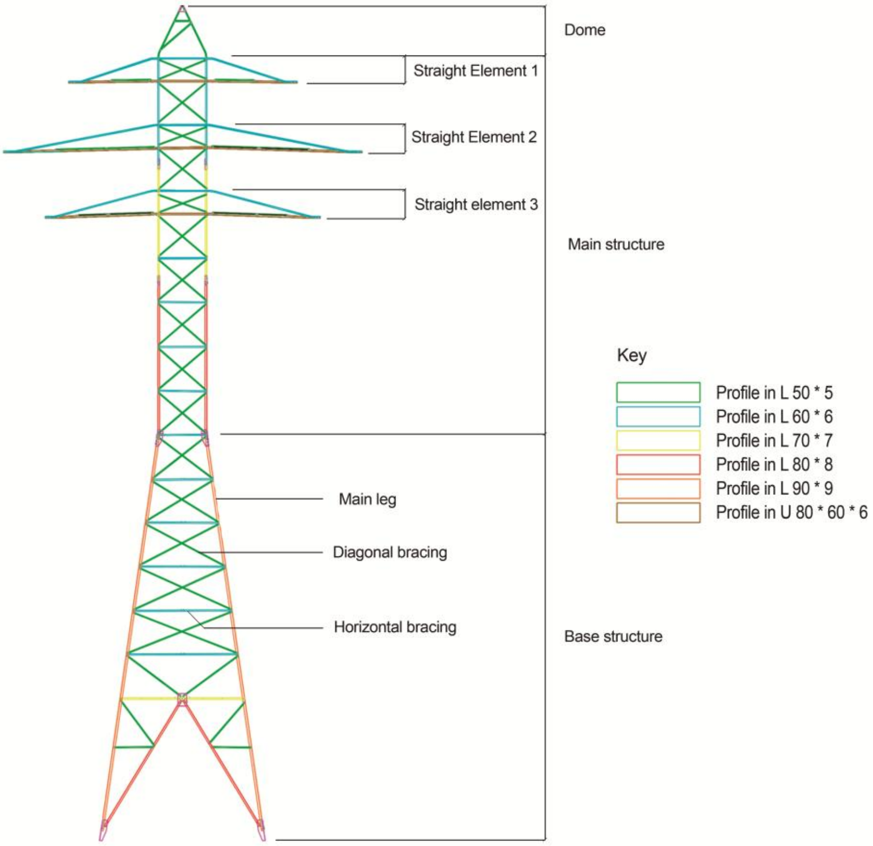

Figure 2. Scheme of main structural parts of transmission tower upon draws of one of the cases studies herein analyzed (Tower 1). Data was obtained from laser scanning technology according to the methodology presented in Section 3.

3. Methodology

3.1. Geometric Digitalization

A laser scanning survey was conducted in order to generate a 3D model accurately describing the structure of the transmission towers. A time-of-flight (ToF) terrestrial laser scanner (TLS), Leica ScanStation2, was used for recording external geometry. This scanner covers a field of view of 360° in the horizontal direction and 270° in the vertical direction, enabling the collection of full panoramic views. The distance measurement is obtained with a nominal accuracy of 4 mm at 25 m range. The horizontal and vertical angular accuracy is of 60 μrad (3.8 mgon). The diameter of the laser spot is 4 mm at 50 m. According to the technical specification of the instrument, it has a maximum acquisition rate of 50,000 points per second. The scanner incorporates a dual axis compensator, so the vertical *Z*-direction is perfectly defined during data acquisition. Due to the complexity of transmission towers and their heights, four scanner stations with a resolution of 5 mm at 25 m were required to enclose the whole structure. The resulting 3D point cloud (about 6 million points per tower) contains geometric data, normally given in Cartesian coordinates (XYZ), as well as the intensity values (*I*). This intensity measurement is referred to as the amount of reflected radiation with respect to the emitted radiation. Typically, this value is normalized in the range of 0–1 or 0–255, 8-bit format in our case. According to the principles of interaction between electromagnetic radiation and matter, the intensity values directly depend on the physical characteristics of the object surface, the wavelength of the incident radiation, and the distance between the laser and the object.

3.2. Geometric Modeling

The geometric modeling of the transmission towers was performed following four steps:

1. Cleaning and segmentation of point clouds in order to remove undesired data, such as reflections, noise or sensor artifacts. This step was performed manually.
2. Alignment of the point clouds from each scan under a common coordinate system. An automated registration method, iterative closest point (ICP) [25], was applied, supported by the identification of matching points and minimizing the Euclidean distance between corresponding point clouds. Initial approximations (three points) were manually identified by the user, trying to guarantee a good distribution around the area of interest and along the three main directions (X, Y, Z). A solid-rigid transformation based on the three points identified was executed. Afterwards, an automatic iterative process to align the different scans was performed taking the Euclidean distance as a minimization criterion.
3. Generating cross-sections and technical drawings of the electrical towers, focusing on the steel profiles that make up each section of the tower and the arrangement of the connections used to define the linkage of these profiles. Different profiles were automatically generated along each main direction (X, Y, Z) in order to obtain vector information of the main sections of the towers and, thus, initial approximations to support the technical drawings and CAD model generation.
4. Obtaining a computer aided design (CAD) model. Since the structural analysis based on a FEM model does not cope with dense laser models, an important step which allows us to pass from the 3D point clouds to a solid geometric model was performed. This step consists in extruding the sections obtained in the step before along its normal direction. Manual interaction is required in this step in order to solve the different intersections between profiles and their connections. In addition, specific existing libraries based on standard steel profiles (*i.e*. L-shaped and U channel profiles) were used for modeling the towers. Geometric modeling was done using Geomagic Spark, 2013 version.

3.3. Structural Analysis

The three towers are formed by angular steel profiles of different dimensions, and given the age of the towers and only for the purpose of the methodology developed in this article, we assume the lower specification for a steel material enabled by [26], type S-235.

This brings the following mechanical properties: Young's modulus of 2.1×10^8 kN/m^2, specific weight of 76.9729 kN/m^3, Poisson's coefficient of 0.3, and yield stress of 235 MPa.

Furthermore, the data of the power line, support type of the tower, and the mechanical characteristics of the electrical drivers are detailed below:

- High voltage power line with rated voltage of 132 kV and 50 Hz AC
- Two duplex-circuit line with alignment support.
- Span: 300 m between supports.

- Electrical driver of aluminum galvanized steel, type LA-280.

The boundary conditions of the three towers are assumed to be articulated supports in each of the legs that make up the outer frame of the towers, so that they have only limited movements according to the global axes (*X*, *Y*, *Z*). The constraints upon the members are explained in each of the structural models discussed above.

The different load conditions are obtained according to [18]. The following descriptions summarize the loads, always bearing in mind that the towers are located in the province of Guadalajara (Spain) and that they are power lines with alignment support and with suspension insulator strings [18].

- Permanent loads: The self-weight of the steel profiles that comprise the towers, electrical conductors, fittings, insulators, and the grounding wire.
- Wind load: It acts upon steel members of the towers, the insulators, and the suspension insulator strings.
- Imbalance of tensile forces: A longitudinal force equivalent to 25% of all unilateral tractions of electrical drivers and grounding wire. This tensile force will be applied at the point where the electrical conductors and the grounding wire are attached to the support, thus taking into account the torsion that these forces could create.
- Electrical conductor failure: A unilateral tensile force related to a single electrical conductor or a grounding cable's failure. The minimum admissible value for the failure is 50% of the broken cable's tension in the power lines that have two conductors per phase.

Taking into account aforementioned load patterns, the current standards [18] refer to certain calculation hypotheses that establish the load cases, shown in Table 1.

Table 1. Load cases considered in structural analysis of towers.

Tower Type	Force Direction	Hypothesis 1	Hypothesis 2	Hypothesis 3
Alignment support and suspension insulator strings	Vertical	Permanent loads, considering the electrical conductors and the grounding cables to withstand wind load according to a 120 km/h wind speed		
	Transversal	Wind load (120 km/h) on electrical conductors, cables, grounding cables and supports of tower	Not applicable	Not applicable
	Longitudinal	Not applicable	Imbalance of tensile stress	Electrical conductor and grounding cable failure

In order to clarify such load cases considered, in Figure 3 is detailed upon finite element model of one of the case studies herein analyzed (Tower 1), the arrangement of the loads in each of the three scenarios previously commented. Similarly, loads are arranged in Towers 2 and 3.

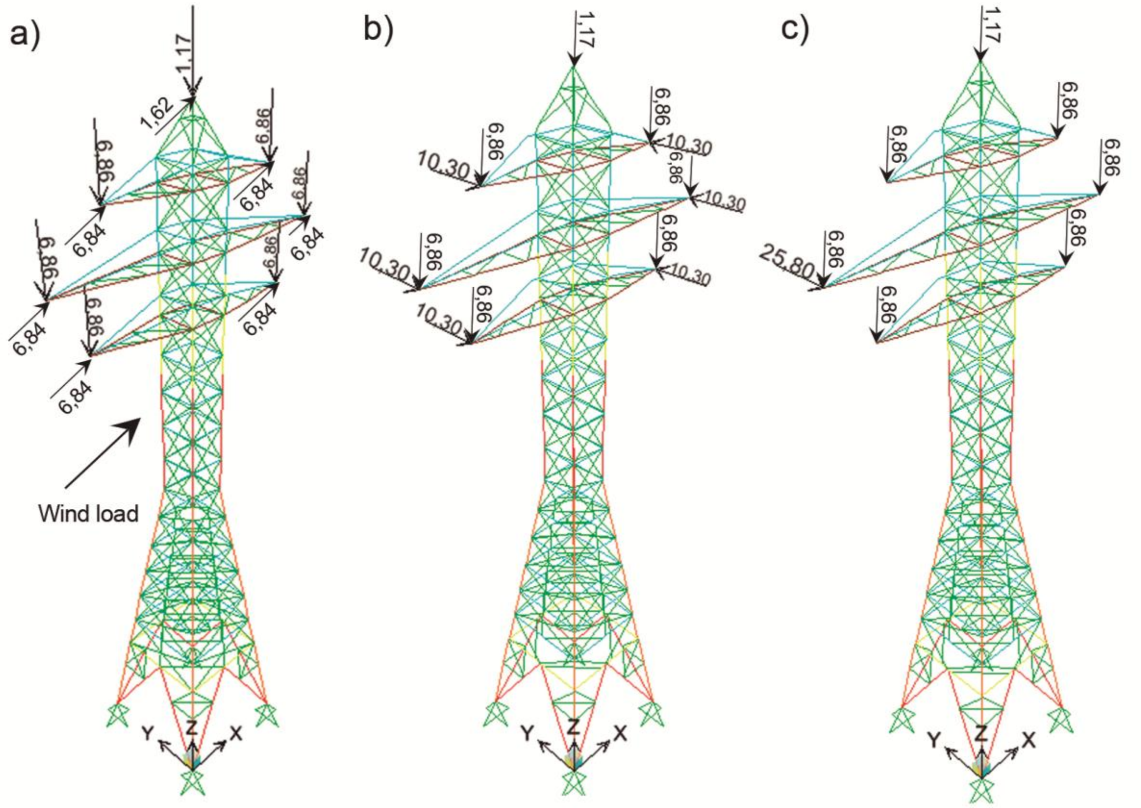

Figure 3. Loads cases considered in structural analysis. (**a**) Hypothesis 1: wind; (**b**) Hypothesis 2: Imbalance tractions; (**c**) Hypothesis 3: electrical conductor failure.

4. Experimental Results and Discussion

4.1. Case Studies

Since this study arises as a consequence of the need to analyze the structural viability of a series of electricity transmission towers that serve as support to an old power electricity line located between the cities of Guadalajara and Torija (Spain), three different cases studies were chosen for the development of the aforementioned methodology.

The electricity transmission towers chosen correspond to a type of tower known as "support alignment" which are disposed over different sections of electricity line. Additionally, in order to extend the study over different structural typologies, each tower corresponds to a different morphology.

The first tower is formed by both a main body support and another principal body (comprising horizontal bracings and diagonal bracings according to a St Andrew's disposition) and three horizontal symmetrical bodies for the support of the cables. The second tower only has a support body (formed by horizontal bracings and secondary diagonal bracings according to a St Andrew's disposition) and three asymmetric horizontal bodies. The third tower is similar to the second one, with exception in the diagonals forming the support body which are not arranged according to a St Andrew's disposition.

Figure 4 shows a photograph of the three towers that composes cases studies described above.

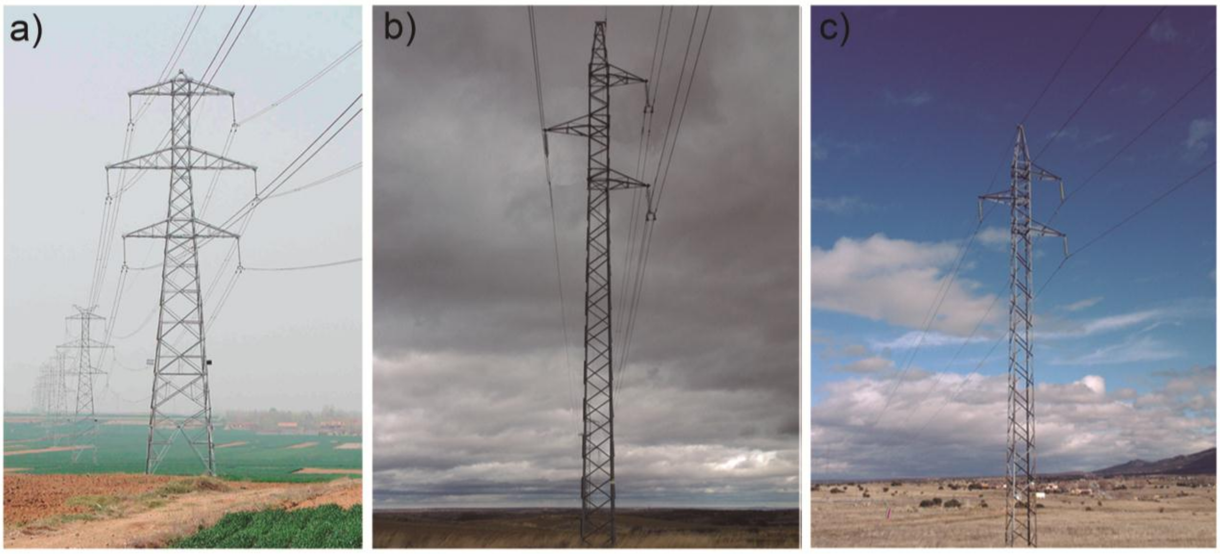

Figure 4. Transmission towers considered in this study: (**a**) tower 1; (**b**) tower 2; (**c**) tower 3.

4.2. Geometric Modeling

Following all steps described in Section 3.2, the point cloud data obtained as a result of a laser scanning survey was subsequently transformed into a CAD model valid for its implementation in the finite element software package SAP2000.

This is a key step required in this kind of reverse engineering process, since data obtained from laser scanning technology do not represent any valid information by itself for the purpose of finite element analysis without suitable data processing [27].

Therefore, taking this into account, CAD models for each one of towers analyzed together with drawings about its current disposition and assembly information were obtained.

Figure 5 shows CAD wire models obtained for each one of towers analyzed. Once such geometrical models were obtained, they were directly imported as a DXF file to SAP2000 software for the finite element analysis stage.

Main geometric data concerns to dimensions of the base, height of the tower, and length of horizontal bodies are displayed in Table 2. The length of the horizontal bodies of the towers is measured from the main body of the tower up to the farthest node. Tower 1 has three horizontal bodies with different dimensions, while in towers 2 and 3, all the horizontal bodies have similar dimensions.

Table 2. Main geometric data for transmission towers analyzed.

Tower	Base Dimensions (m)	Height (m)	Length of Horizontal Body (m)
1	7.25 × 7.25	37.25	3.95 6.25 5.0
2	1.5 × 1.5	18.50	2.0
3	1.5 × 1.5	18.50	2.15

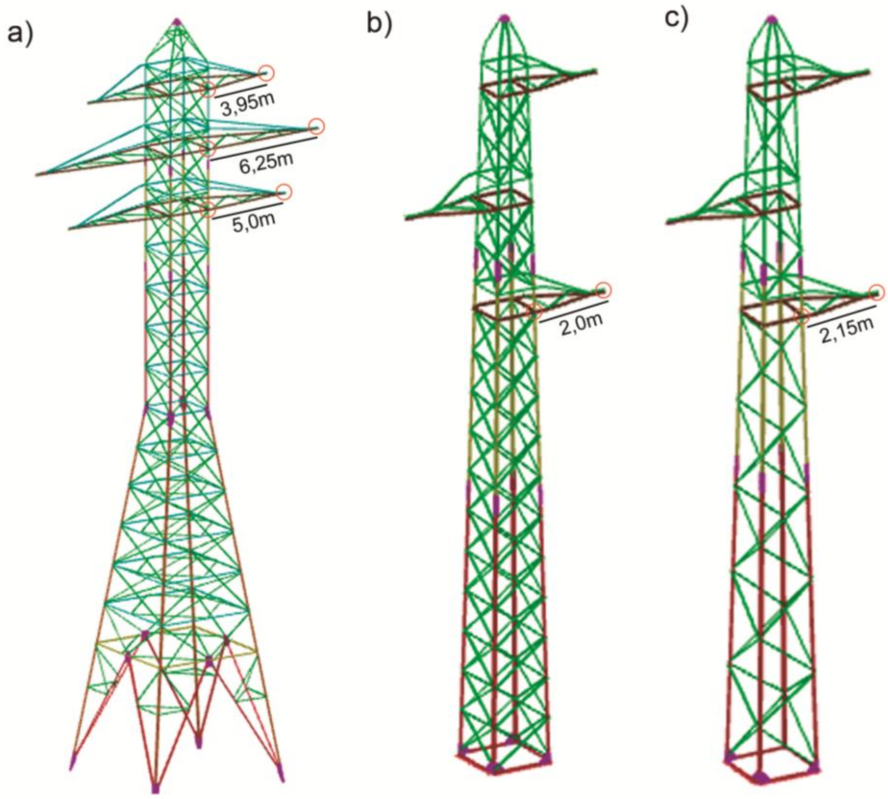

Figure 5. Geometrical CAD wireframe models of transmission towers analyzed: (**a**) tower 1; (**b**) tower 2; and (**c**) tower 3.

4.3. Structural Analysis

As was previously indicated, finite element models of transmission towers were analyzed in SAP2000 software. Within this package, frame elements were chosen so that stiffness against rotations could be considered in all of those nodes assumed to behave as rigid joints. For all other cases where moments will not be considered (as, for example, in diagonal members pin jointed to truss chords), releases end options in nodal degrees of freedom could be imposed for transforming frame elements to truss elements and, thus, only axial forces be considered. For all structural models herein developed, analysis was carried out considering linear elastic behavior.

Table 3 shows the number of frame elements and the number of degrees of freedom for each of the finite element model developed, taking into account the type of transmission tower structure and structural model approach.

Table 3. Number of frames elements and degrees of freedom for each one of the three different structural models considered upon cases studies analyzed.

Tower	**Model 1**		**Model 2**		**Model 3**	
-	**Frame Elements**	**Degrees of Freedom**	**Frame Elements**	**Degrees of Freedom**	**Frame Elements**	**Degrees of Freedom**
1	419	3320	419	3896	683	6772
2	241	1909	241	2348	321	3308
3	181	1448	181	2100	226	2408

4.3.1. Displacements

In this section, displacements [12,14,28] experimented by each tower under different load cases and structural models are analyzed and discussed.

To carry out the analysis and comparison, representative points of the three towers associated with the nodes of the horizontal bodies and the upper node of the dome were selected. Figure 6 shows an example of these nodes together with the displacements experimented for each tower under Loading Case 1 (wind load) and structural model considering real geometry for upper node of the dome (Node 1).

Table 4 reports both the values of maximum displacements obtained for Model 3 (real geometry) and the node where they take place (one of the aforementioned) for the different loading scenarios considered in each one of the transmission towers analyzed. Table 5 represents a comparison (in absolute value) between maximum displacements for Models 1 and 2 with respect to Model 3 (considered as the most accurate).

In both tables, the structural model considered is represented by "M" followed by a number indicating the corresponding model described in Section 2. Load cases considered are indicated by "H" denoting each one of the hypothesis described in Section 3.3 and Table 1 is applicable.

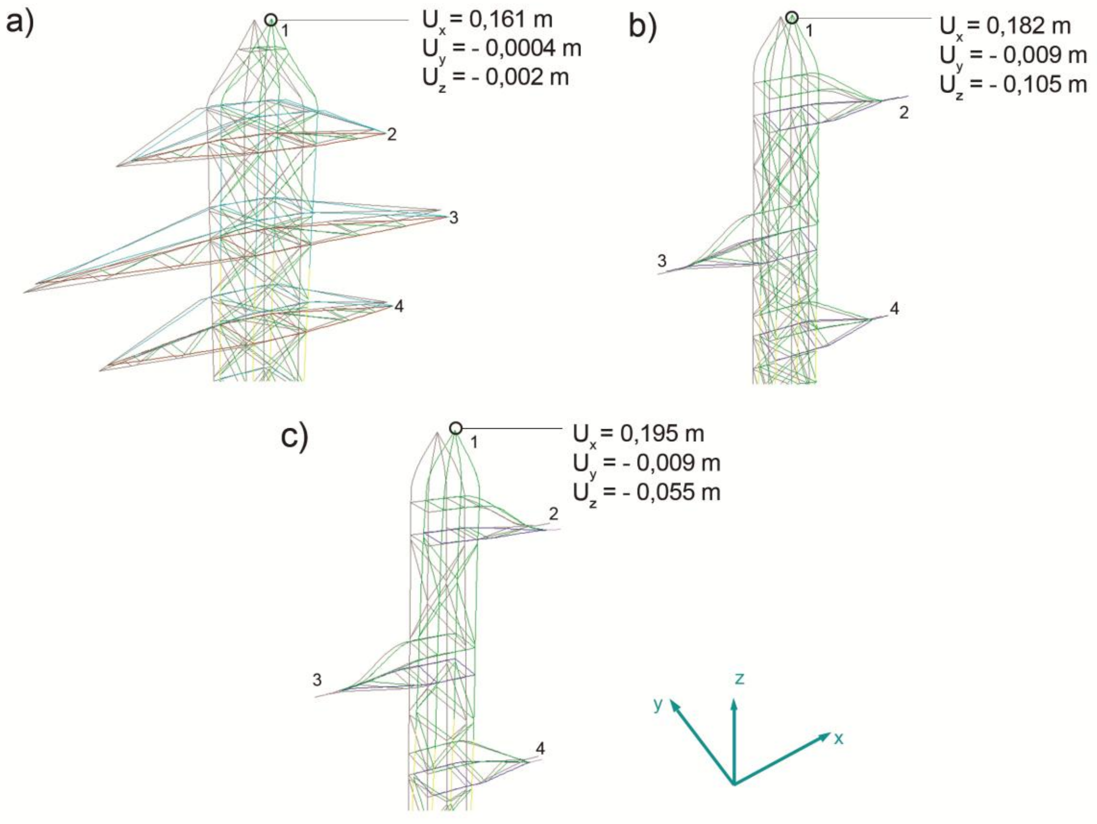

Figure. 6. Nodes considered for analysis of displacements according to the global axis directions in each tower and structural model analyzed. (**a**) tower 1; (**b**) tower 2; (**c**)tower 3.

Table 4. Values of the maximum displacements experimented for Model 3 in all transmission towers and load cases considered with the indication of the node where they take place.

Tower	Model	Displacement Ux (mm)/Node			Displacement Uy (mm)/Node			Displacement Uz (mm)/Node		
		H1	H2	H3	H1	H2	H3	H1	H2	H3
1	M3	161/1	14/3	12/4	8/3	162/3	140/3	132/3	8/2	8/2
2	M3	182/1	44/2	11/2	9/1	88/1	125/3	123/2	11/2	15/2
3	M3	195/1	52/2	15/2	9/1	97/1	146/3	85/2	21/3	18/2

Table 5. Comparison between displacements experimented by Models 1 and 2 with respect to Model 3 in all transmission towers and load cases considered.

Tower	Models Compared	Maximum Deviation Ux (mm)/Node			Maximum Deviation Uy (mm)/Node			Maximum Deviation Uz (mm)/Node		
		H1	H2	H3	H1	H2	H3	H1	H2	H3
1	M3 front M1	93/1	6.3/3	4/4	5/3	52/3	62/3	27/3	4/2	4.6/2
	M3 front M2	95/1	6.3/3	4/4	5/3	65/3	78/3	29/3	4/2	5/2
2	M3 front M1	66/1	8.3/2	7/2	4/1	44/1	39/3	35/2	4/2	9/2
	M3 front M2	80/1	9/1	8/2	6/1	43/2	48/3	38/2	5/2	10/2
3	M3 front M1	75/1	9/2	6/2	7/1	38/1	37/3	38/2	3/3	5/2
	M3 front M2	65/3	7/1	7/2	4/2	32/2	42/3	43/3	5/3	6/3

The results displayed in Table 5 show that principal differences always take place in all towers under Hypothesis 1 (wind load case). The differences found between Model 3 and Model 1, reaching 95 mm in the global *X* direction is remarkable.

There are also significant differences for all towers, although less than aforementioned for displacements of nodes in the global *Y* direction under Hypotheses 2 and 3 (Imbalance tractions and cable break). The reasons that can explain these differences may be due to the following factors:

1. Consideration of the initial small vertical deviations of the towers' main bodies along with the application of wind forces in Hypothesis 1, precisely in that direction, may have accentuated the displacements in global *X* direction obtained in Model 3.
2. The fact of considering the true connections of the profiles onto the nodes (misalignment) has a drastic consequence upon the behavior of the structure since it directly affects to its stiffness.

In Models 1 and 2 "ideal nodes" connecting various elements mobilize stiffness of all concurrent elements (truss chords, diagonal, and horizontal members): axial stiffness in Model 1, and axial and bending stiffness in Model 2; in Model 3, however, it does not occur equally.

When considering real geometry, the existence of "intermediate" nodes inserted into the truss chords cause local bending moments and, thereby, additional rotations that accentuate local deformations of the structures. Therefore, we can state that the improper execution of the connections leads to a less stiff structure and may be the main cause of the differences found between the displacements obtained in the three towers for global *X* direction under Hypothesis 1 and for global *Y* direction under Hypotheses 2 and 3.

A representative example of this behavior can be seen in Figure 7, which shows the deformed shape of tower 1 for Models 2 and 3 under Load Case 3 (electrical conductor break), which subjects the body of the tower to a torque.

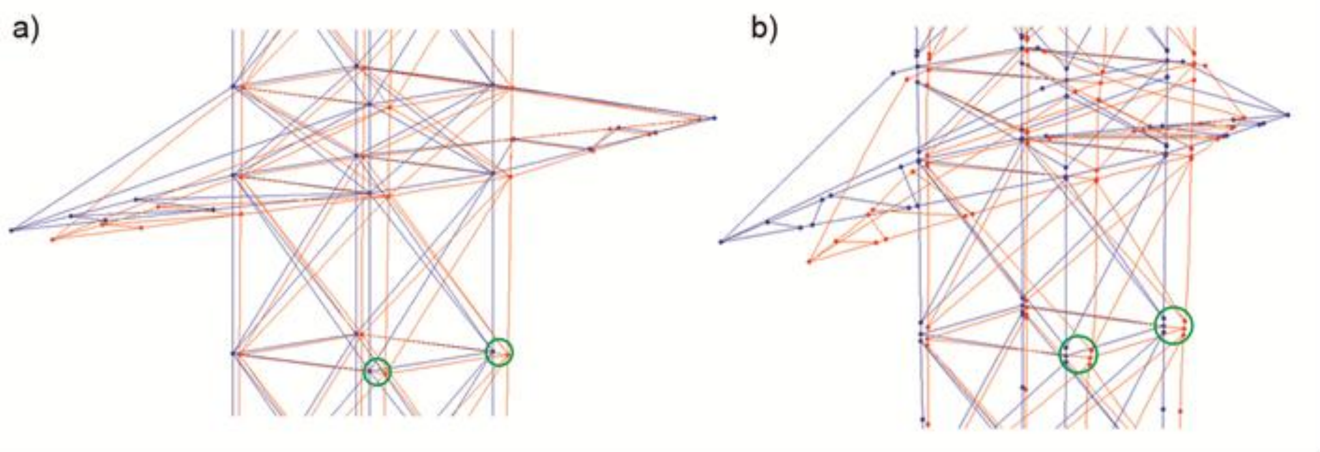

Figure 7. Detail of the different structural behavior of the Tower 1 under Load Case 3 in Model 2 (**a**) and Model 3 (**b**). Undeformed shape is shown in blue and deformade shape in red.

4.3.2. Stresses in Structural Elements

Finally, a comparison regarding the stresses in structural elements [12,14,15,16,29] is also established. Table 6 shows, based on the finite element results obtained, the maximum normal stresses for each transmission tower and its respective structural model. Particularly, it details for each tower, and for each model, the maximum normal stress in which member it occurs, and under which loading case is developed.

Table 6. Structural elements with maximum normal stresses for all transmission towers analyzed and structural models considered.

Tower	Model	Maximum Stress (Mpa)/Frame Element		
		Hypothesis 1	**Hypothesis 2**	**Hypothesis 3**
	1	215.15/90	165.26/401	195.56/468
1	2	225.36/90	170.23/401	205.21/468
	3	252.30/696	185.96/433	230.20/696
	1	218.96/380	155.50/442	221.32/405
2	2	222.56/380	163.89/442	232.45/405
	3	234.21/178	181.78/439	245.63/189
	1	222.25/272	167.25/353	221.45/107
3	2	232.63/272	175.63/353	226.12/107
	3	252.58/272	195.56/353	233.16/104

Based on Table 6, it can be observed that the differences in maximum normal stresses between Model 1 and Model 2 for all towers are quite small and the maximum of these values are always produced in the same members for both models. On the contrary, significant differences appear when compared to Model 3. Consider the case of Tower 1 and under Load Case 1 where the differences is approximately 37 MPa.

Moreover, it could be observed that the maximum stresses in Model 3 no longer occur at the same members that for Models 1 and 2. This is due to the singular arrangement of nodes in Model 3 (members do not intersect at a single point) which leads to a different discretization of frame elements compared to Models 1 and 2.

Rising stresses in Model 3 are mainly due to secondary stresses caused by the additional bending moments derived from small geometric eccentricities at diagonal and horizontal member's connections upon truss chords which are neglected in Model 1 and Model 2.

The higher the eccentricity is at the connections, the greater the induced bending moments will be. Likewise, the level of induced secondary stresses will be influenced by the stiffness of the truss chords; the greater is their stiffness the greater bending moments will be induced.

This is perhaps the cause of the observed differences for Tower 1 under Load Case 1, where the stiffness of the truss chords relative to the overall stiffness of the whole structure is greater than in the other two towers, thus providing greater increased stresses (37 Mpa) with respect to the theoretical Model 1.

It should be also noted that for Model 3, under certain load cases in all towers, some elements exceed the yield point of the material. This circumstance is highlighted in Figure 8, where for tower 1 and under Load Case 1, the element with maximum normal stress reaches 252 MPa, exceeding the elastic limit of the material in 17 MPa. Figure 8 also shows a representative image of the above discussed; the effects of improper execution of nodes upon truss chords and, consequently, the different discretization of frame elements in the model with ideal geometry and those based on real geometry.

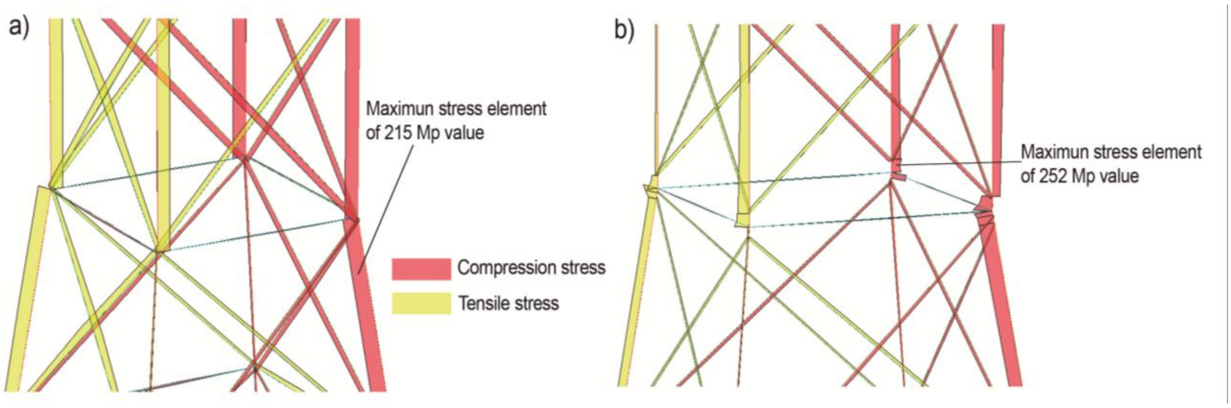

Figure 8. Normal stresses distribution onto Tower 1 for different structural models. (**a**) Model 1, with ideal representation of the nodes and maximum stress below the yield limit of steel; (**b**) Model 3, including the improper executions of the nodes and maximum stress exceeding the yield limit of steel.

As for results involving values of maximum normal stresses, they should be analyzed carefully. Due to the age of the structures, assumed material properties have been chosen according to the minimum value specified by the current regulatory codes; however, these values do not have to match the actual properties of the structure. Accordingly, appropriate experimental tests should be carried out in order to improve their characterization and, thus, derive proper conclusions about the real current safety state of the structure. Note, also, that the structural analysis of transmission towers was carried out assuming a linear elastic behavior.

Nonetheless, the results herein obtained could be considered as acceptable, bearing in mind that the present work is focused in defining an overall methodology able to detect the geometric imperfections present in electricity transmission tower structures by means of precise laser scanning systems and the procedures to incorporate them in structural models based on finite element methods.

5. Conclusions

Terrestrial laser scanning enabled performing a non-invasive remote survey of different transmission tower structures; it should be noted that these are objects of great complexity, not only for their size, but also by their geometry and their high heights. This technology allowed detecting significant imperfections in terms of connections between the members at the nodes, loss of verticality of the towers and the lack of horizontality in the horizontal bracings. The aforementioned imperfections motivated the consideration of different structural models for the towers, in order to analyze how this affects their structural behavior. To that purpose, three different models have been carried out: first, a model with an ideal geometry and considering perfectly pin jointed nodes (supposed model in original calculations of the structures); second, a model with ideal geometry, but taking into account real existing continuity in some profiles by means of rigid connections and pin jointed connections of all others elements upon them; and finally, a third model similar to that previous, but with a real geometry incorporating all imperfections obtained from laser scanning data.

When analyzing the results obtained in terms of displacements and stresses yielded by the different structural models considered, significant differences were observed. At those nodes considered for the comparison of displacements, differences between models with ideal geometry, and models with real geometry reached several centimeters, becoming the highest value (for Node 1 in Tower 1 under wind loading) 9.5 cm respect to the X global direction.

The study of stresses brings some other conclusions. Considering models with ideal geometry and considering pinned joints (Model 1) or rigid joints (Model 2) no significant differences were found. Indeed, maximum normal stresses in elements always take place in the same elements for both models. On the contrary, the model which accounts for real geometry (Model 3) presented notable differences when compared with their respective idealized models. Notorious increases in stresses were detected under certain loading conditions, even reaching the elastic limit of the steel in some occasions.

Differences observed in displacements, stresses in elements and, thereby, whole structural behavior of towers suggest that a detailed survey and conscientious structural analysis has to be carried out when these type of structures will be required for future uses as, for example, new communication services that increase their service loads.

Further studies could contemplate performing nonlinear analysis to extend and improve the results herein obtained, either by considering geometric nonlinearity effects such as P-Delta effects and plastic behavior of steel material. Moreover, due to the nature of the structures (quite slender, and with very slender members) the issue of structural stability should also be addressed. Therefore, it is expected that the combination in the use of the information already available, with the procedure herein developed, together with the consideration of more advanced topics related to strength and structural stability evaluation, will bring a deep insight in the behavior of the towers.

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Author Contributions

All authors contributed extensively to the work presented in this paper.

Conflicts of Interest

The authors declare no conflict of interest.

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