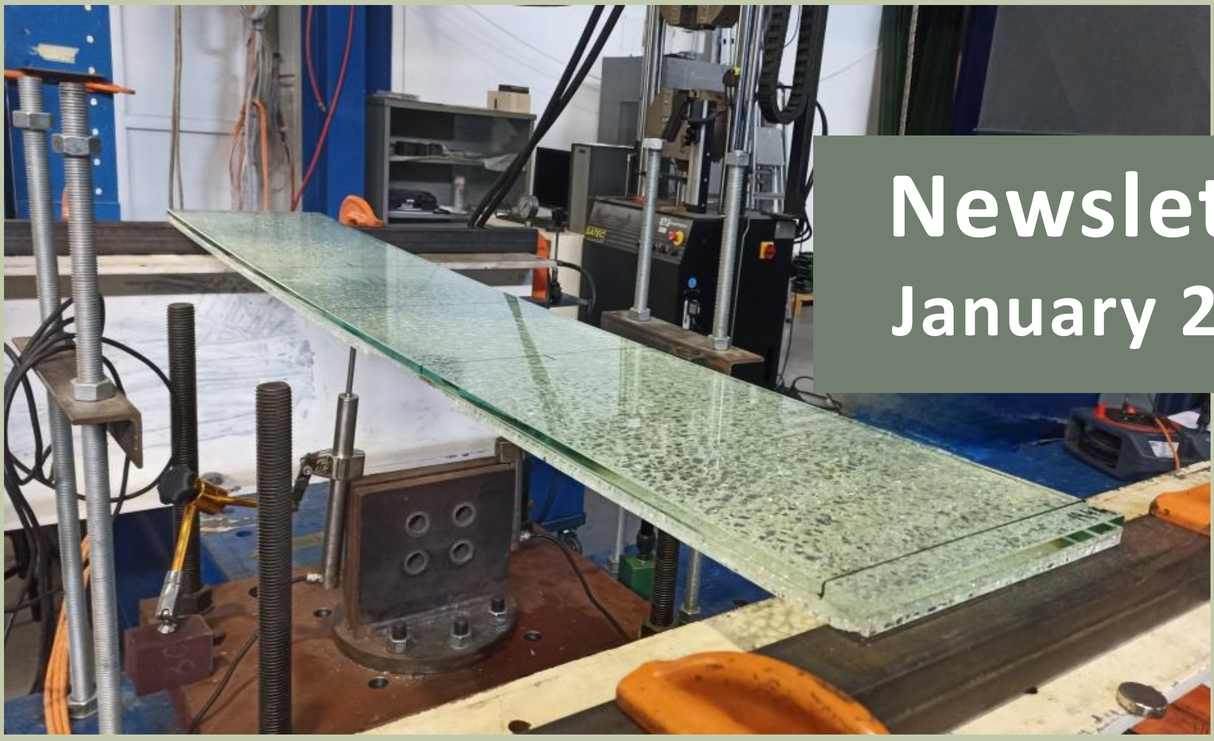
ISS

Institute of Steel Structures



Newsletter January 2022

National Technical University of Athens
School of Civil Engineering

Table of contents

Institute of Steel Structures - NTUA / January 2022

Designed & formatted by: **Aikaterini Michaltsou**

Front cover: **Laminated glass panels tests**

Welcome to the new issue of ISS-NTUA Newsletter!

Dear readers,

While the world continues to struggle against the Covid-19 pandemic, at the Institute of Steel Structures of the National Technical University of Athens we have continued doing our best to serve the Greek and international steel structures community in education and research.

The recent period was marked by the loss of Professor Emeritus George Ioannidis, a distinguished member of ISS and former president of the Hellenic Steel Structures Research Society. You may find a memorial statement in the following pages.

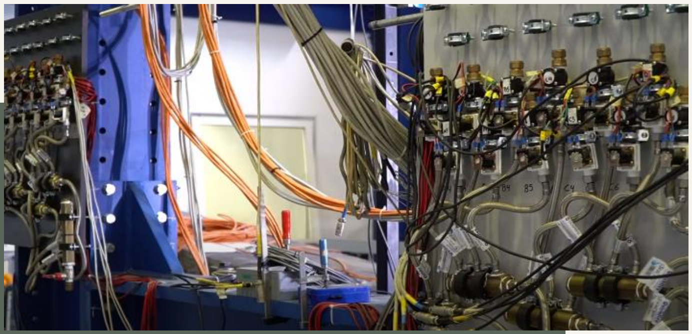
Amid planned inspection and maintenance activities of the steel structures at the Olympic Athletic Center of Athens, Professor Emeritus John Ermopoulos remembers his own and our Institute's contribution to their design and construction before the 2004 Olympic Games.

Dr. Thanopoulos reports on recent research regarding lateral stability analysis of composite plate girder bridges, while Dr. Lignos, Ms. El Dahr and Prof. Vayas describe part of an ongoing effort to develop and apply low-cost sensors for structural health monitoring. Dr. Melissianos and Prof. Vamvatsikos discuss recent progress on evaluating the design fault displacement for lifelines at fault crossings. Finally, the recently submitted doctoral thesis of Mr. Bezas is outlined, on new design rules for steel members with angle sections, carried out in the framework of European research project ANGELHY.

The Newsletter concludes with a photo of our Institute during the recent snowstorm that hit Athens, a rather rare occurrence for our city.

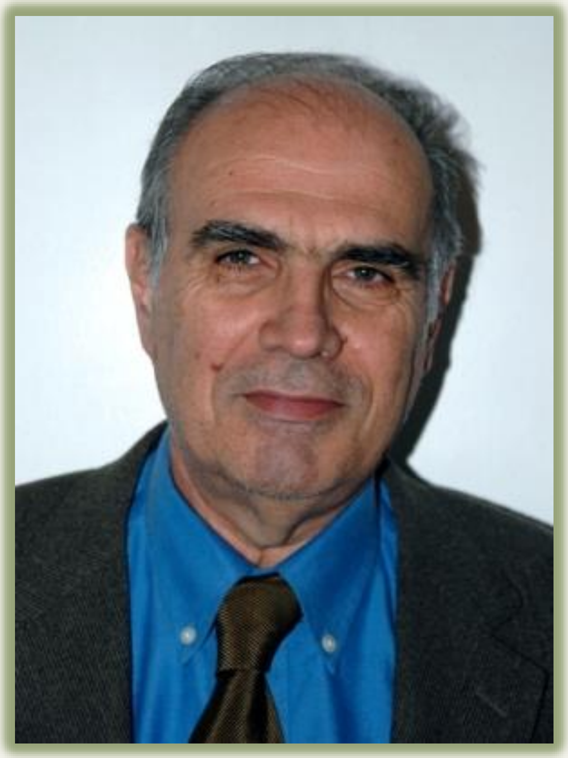
Starting 2022 we look forward to a productive year with fewer difficulties and more inspiring and challenging engineering endeavors, and we wish you all the same.

Charis Gantes



George Ioannidis

16 January 1946 – 29 November 2021



The members of the Institute of Steel Structures were informed with deep sorrow that our distinguished colleague and friend George Ioannidis, Emeritus Professor of the School of Civil Engineering at NTUA, passed away on 29 November 2021.

George Ioannidis was born in Athens in 1946 and graduated from the Varvakeio Model School in 1963. He studied at the School of Civil Engineering of NTUA from 1963 to 1968 and then at the Higher Institute of Reinforced Concrete of Marseille with a scholarship from the French Government. His academic career at NTUA began in 1972 at the Chair of Statics and Steel Bridges and continued until his retirement as Professor of the Institute of Steel Structures in 2013.

At the same time, he had an exceptional professional activity as structural designer or consultant of a wide range of projects, comprising energy, industrial and sports facilities, shipyards, airplane hangars etc., including structures for the Eleftherios Venizelos Athens Airport and the 2004 Athens Olympic Games. Photos of some of the structures he designed are shown on the next page.

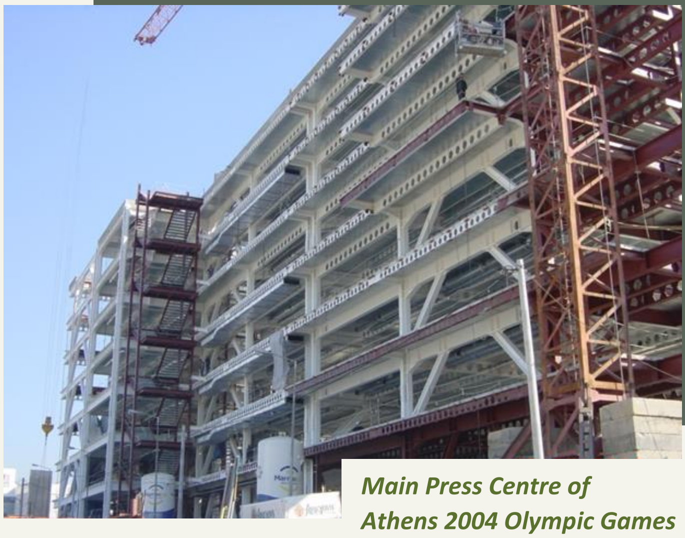
He was an enlightened and beloved teacher of metal structures of a whole generation of Greek civil engineers, to whom he transferred generously his knowledge and practical experience, teaching them how to solve practical problems by combining theoretical knowledge with engineering judgement while respecting technological limitations, adapting his teaching to the level of each audience.

George Ioannidis was a fair, noble and sweet-spoken man of low tones, whom all those who were lucky enough to come across him will remember with great respect and love.

by John Ermopoulos, Ioannis Vayas and Charis Gantes

Hangar of Olympic Aviation at Eleftherios Venizelos Airport of Athens

George Ioannidis inside the Olympic Aviation Hangar



Main Press Centre of Athens 2004 Olympic Games

Hellenic Navy Ship Maintenance Hangar

Galatsi Olympic Center

Flying ash facility of Hellenic Public Power Corporation, Ptolemais

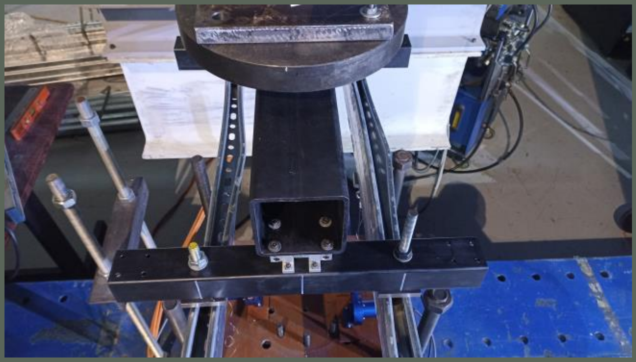
LECTURE

The Institute of Steel Structures at NTUA in cooperation with the Steel Structures Research Society continued the tradition of organizing lectures addressed to students and practicing engineers. In December 2021, Prof. Spyros Karamanos of the University of Thessaly presented “Deep-Water Mission to Venus - The Technological Challenge of Underwater Pipelines”.

THIN-WALLED PURLIN TESTS

A series of four-point bending tests of thin-walled U-shaped purlins used to support photovoltaic panels has been carried out. The tests covered 1.5mm and 2mm thick sections and all loading directions.

The main objective was to investigate the influence of slotted holes, drilled to facilitate the installation process, on the bending response of the purlins. For that purpose, purlins with and without slotted holes were tested. It was deduced that holes in different arrangements have a very small effect on the measured response.



LAMINATED GLASS PANELS TESTS

A series of tests of two types of secure laminated glass panels were performed, aiming at studying their response to transverse loading and the local behavior at their supports. The dimensions of both types were 1500mm x 250mm.

The first type consisted of two 12mm thick glass panels with a 1mm thick membrane in between, while in the second type the two glass panels were 10mm thick. Four-point bending tests were carried out, with 25mm wide supports at the two edges and concentrated loads applied by a hydraulic actuator through a system of hinged spreader beams for balanced application. Measurements were taken using a 300kN load cell and two ±75mm LVDTs. Linear elastic response was observed, followed by brittle fracture of the glass panel in tension. Glass fragments were smaller than 10mm and limited debonding from the protective membrane was observed.

PUBLICATIONS

In 2021 the members of the Institute have published 26 papers in international journals. For a full list of publications please visit:
http://labmetalstructures.civil.ntua.gr/cms/en/research/publications

Steel structures for the 2004 Athens Olympic Games

In the period before the 2004 Athens Olympic Games, there was an "explosion" in the use of steel in Greece, as the need for covering long spans, good seismic performance and speed of erection resulted in adopting steel for many new spectacular structures (sport halls, buildings etc.) that were planned, designed, and constructed mostly by Greek architects, engineers, contractors, and manufacturers. Faculty members of the Institute Steel Structures played an important role in that process, participating in many of these projects from different positions. In the following, some of these structures that were erected in the Olympic Athletic Center of Athens (OACA, Fig 1) are briefly presented.

Fig. 1 The OACA area

ROOF OF THE OLYMPIC STADIUM

The steel roof of the Olympic Stadium (Figs 2 to 5) is composed of two pairs of arches, which are connected at their support points and are lying on a common vertical plane. A series of transverse beams carrying the polycarbonic sheets of the cladding system are supported on the lower arches, while prestressed vertical and inclined cables connected to the upper arches, suspend the lower arches and the transverse beams. The total steel weight of the superstructure is 18000 tons. The arches span 305 m, while the height of the upper arches is 70 m. The diameters of the upper and lower arches are 3.20 m and 3.60 m respectively, with wall thickness between 60 and 100 mm. The foundation at each of the four support points consists of a group of piles with 1.50 m diameter and 25-30 m length. Because of parallel works in the interior of the stadium, the two subparts of the roof (i.e., each pair of arches with the cladding) were erected at a distance from the stadium and were then moved to their final position by sliding (about 70 m with a speed of 3 to 5 m/hour).

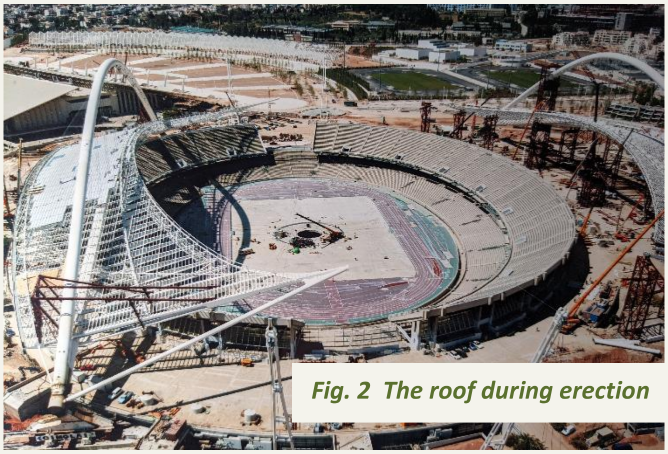


Fig. 2 The roof during erection

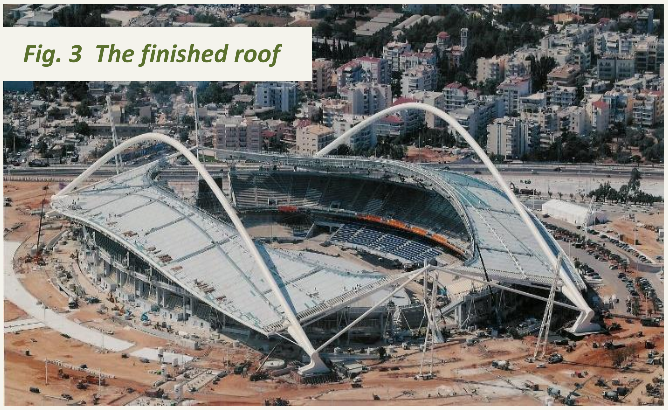


Fig. 3 The finished roof

Fig. 4 Internal view

Fig. 5 Roof support

VELODROME

The steel roof of the Velodrome (Figs 6 to 8) is supported by two pairs of inclined arches welded together at their ends and has a total weight of 4500 tons. The cladding consists of white steel sheets externally and timber internally except for a central strip of the roof covered by blue polycarbonic sheets. The longitudinal span between the supports is 145 m, the maximum width of the roof is 106 m, and the height is 46 m. The main beams of the roof are placed transversally and supported by the lower arches (with diameter 1.80 m and thickness 32 to 54 mm). Prestressed cables with 50 mm diameter are used to suspend the roof from the upper arches (with diameter 1.30 m and thickness 60 mm) and connect the arches.

Fig. 6 Erection of Velodrome

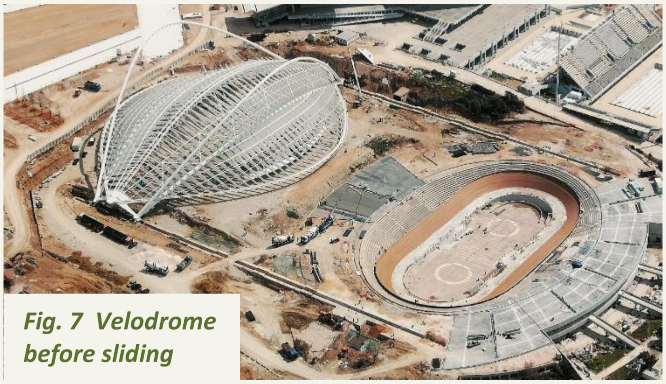


Fig. 7 Velodrome before sliding

Because of parallel work in the area of the velodrome, the whole roof was erected at a distance of 135 m from its final position and was then moved by sliding (with a speed of 10 m/hour).

Fig. 8 Completed velodrome

AGORA

The AGORA (Figs 9 to 10) is a multi-use recreational area consisting of fixed I-section parabolic arches of 26 m span and two different heights, i.e. 19 m and 22 m, spaced at 5 m. The whole length of the curved AGORA is approximately 450 m. The arches are connected by inclined bars (RHS and angles) to provide partial shading of the covered space and to secure the stability of the structure.

Fig. 9 AGORA under construction

Fig. 10 Completed AGORA

NATIONS' WALL

The Nations' wall (Figs 11 to 13) is a 260 m structure comprising eleven inclined tapered steel columns with a height of 15 m and a horizontal continuous beam at their top (1.75 m x 1.00 m). 960 vertical hollow section bars with 20 m length are connected on this beam.

Fig. 11 Nations' wall under construction

Fig. 12 Finished Nations' wall

By utilizing a mechanism lying on the upper part of the beam these bars can be rotated in a sinusoidal form, (Fig. 13). The foundation consists of separated pile caps supported on four piles for each column.

Fig. 13 Rotating Nations' wall

MAIN ENTRANCE CANOPIES

The Main Entrance Canopies (Figs 14 to 17) are supported by a pair of arch tubes (1 m diameter, 20-30 mm thickness), fixed on concrete pile caps with eight piles below each pile cap. The height of the structure is 10 m, the span of the arches is 54 m and the cantilever span is 44 m.

Fig. 14 Arches during fabrication

Fig. 15 Canopy under construction

Fig. 16 Canopy under construction

The canopies are covered by fabric membrane between the arches and metal louvers over the cantilever, to offer shadow to the visitors.

Fig. 17 Finished canopy

by John Ermopoulos

Lateral stability analysis of composite plate girder bridges

The bending capacity of plate girders in bridge structures may be highly reduced by global instability phenomena, so that lateral torsional buckling (LTB) becomes important in conceptual, final and execution design. Depending on the geometric and support conditions, composite plate girder bridges are susceptible to lateral instability both at construction and at service stage. At construction stage, the top flange is not yet connected to the concrete deck so it may deflect laterally in the span region of both simple and continuous systems. At service stage, only the bottom flange at the internal supports is susceptible to LTB since the lateral deflections of the top flange are restrained by the deck.

The purpose during the conceptual design is the development of an adequate bracing layout for the compression flanges so that LTB will not excessively reduce the bending capacity of the girder at all stages of construction. Typical configurations for bridge construction consist of horizontal and/or vertical bracings. Based on the characteristics of the bracing system, two basic types may be distinguished: one that provides fixed lateral support and one that provides flexible lateral support. In both cases the lateral support may be at discrete points or continuous along the length of the beam.

Due to the importance of the LTB phenomenon, various methods are provided in the relevant codes. As a simplification of the beam method that is generally proposed in EN1993-1-1 [1] for the design of uniform members in bending, EN1993-2 [2] allows for the compression flange of bridge girders to be modelled as an equivalent member under compression. In addition, the so-called general method defined in [1] can be universally used for the verification of the resistance to lateral and lateral torsional buckling for structural steel components.

In recent work carried out by ISS researchers, all afore-mentioned methods are critically reviewed, especially regarding their applicability for steel and steel/concrete composite bridges [4].

In addition, typical bridge girders are analysed with a truss model introduced by Vayas et al. [3], which can account for the web flexibility, existence of web stiffeners, change of cross section etc. Elastic critical loads and design resistances are determined using the general method for various bracing configurations at construction and service stages. The results are compared with the ones derived from the relevant provisions of [2].

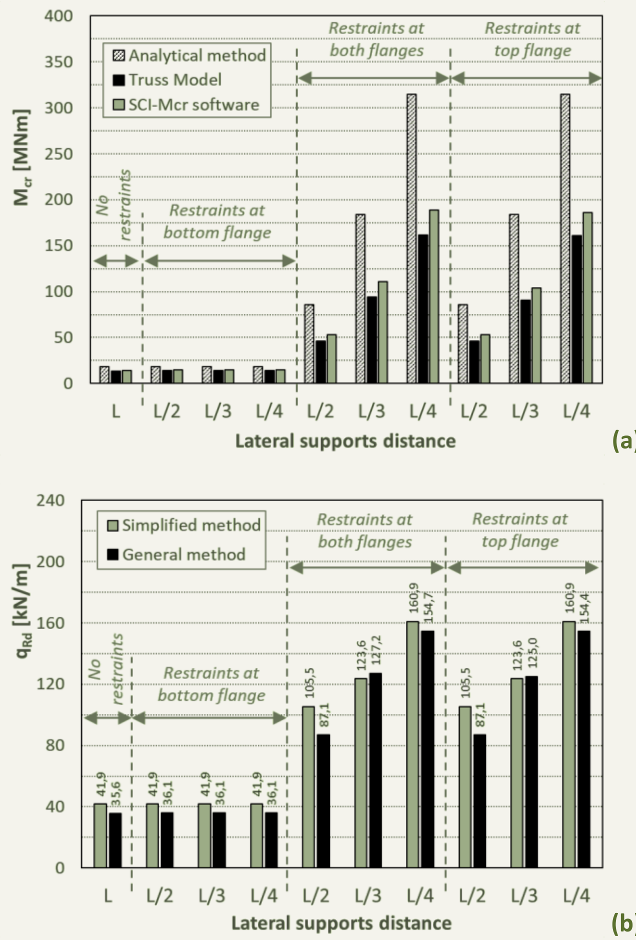


Fig. 1 Simply supported beam – H = 2500 mm: Comparison of (a) elastic critical moments M_{cr} and (b) design loads against LTB q_{Rd} for various restrain conditions and analysis methods

This is performed for two beams, one with typical buildings dimensions (height 300 mm) and one for bridges (height 2500 mm). Representative results are shown in Figure 1. Interesting observations can be made:

- The analytical calculation of the elastic critical moment of the deep girder can significantly overestimate its value when compared to the calculation of more accurate, specialised software. The effect of this phenomenon increases as the distance between lateral supports decreases.
- As far as the design LTB loads are concerned, the simplified method is conservative in the case of the compact beam with no restraint. This is due to the fact that this method ignores the beneficial influence of the tension flange through the bending stiffness of the web. As a result, this effect is reduced for smaller buckling lengths, which leads to better agreement of the two methods.
- For the deeper beam, the influence of the web becomes practically insignificant due to its flexibility, so the two methods give practically identical results.

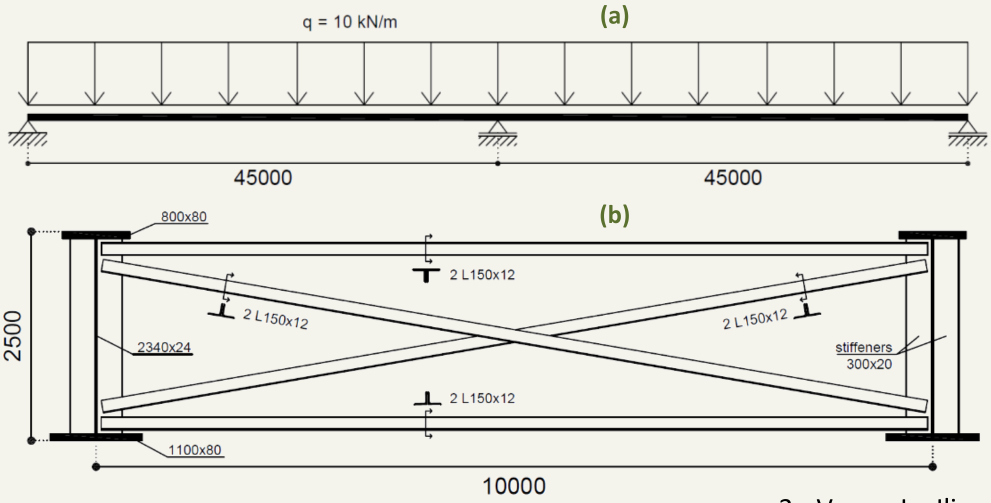


Fig. 2(a) Longitudinal layout and (b) cross section of the twin-girder two-span bridge

Having demonstrated the validity of the truss model for a well-documented case like the simply supported beam, the slightly more complex case of the two-span bridge of Figure 2 is investigated. The bridge is a typical two-girder composite bridge and is examined during concreting, in order to investigate the buckling of both flanges (Figure 3), as well as during its service life. Various layouts of vertical cross girders and web stiffeners are examined. Once more, the results are compared with the ones given by the codes and the validity of the truss model is demonstrated.

REFERENCES

1. EN1993-1-1: Design of Steel Structures – General Rules and Rules for Buildings, CEN, 2005.
2. EN1993-2: Design of Steel Structures – Steel Bridges, CEN, 2006.
3. Vayas I., Iliopoulos A. and Adamakos Th.: Spatial systems for modelling steel-concrete composite bridges – comparison of grillage systems and FE models, Steel Construction Design and Research 3 (2010), 100-111.
4. Thanopoulos P., Louverdi S., Spiliopoulos A. and Vayas I.: Lateral Stability Analysis for Composite Plate Girder Bridges. International Journal of Steel Structures 21, 430-441, Springer, 2021.

All images and content have been taken from [4]. Full publication and extensive results can be found in:
https://doi.org/10.1007/s13296-020-00446-x

by Pavlos Thanopoulos

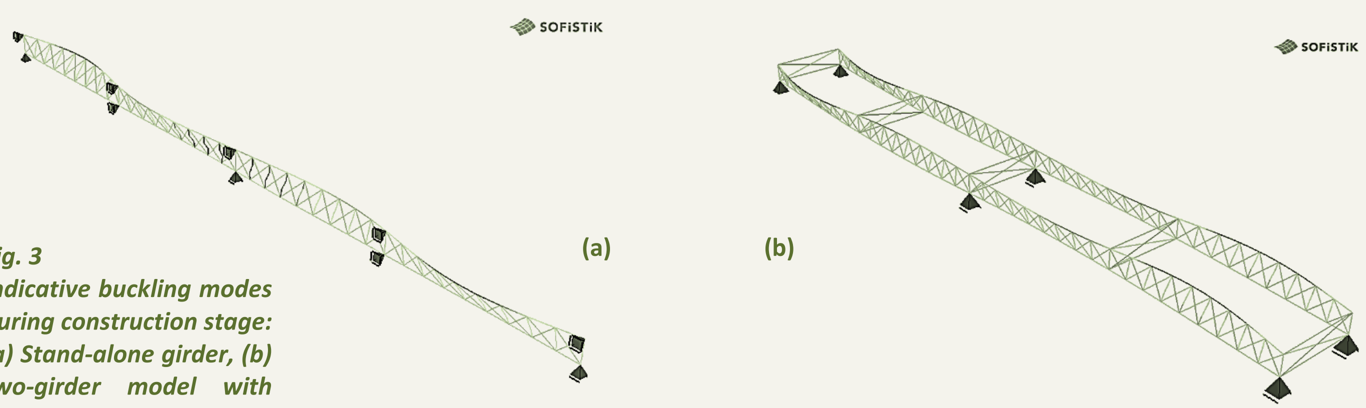


Fig. 3
Indicative buckling modes during construction stage: (a) Stand-alone girder, (b) two-girder model with bracings

Design and validation of an accurate, low-cost data acquisition system for structural health monitoring purposes

Structural health monitoring is an effective technique devoted to enhance the robustness, and validate the safety of an infrastructure, by assessing its health condition and detecting anomalies and damages.

In this work a microcontroller based, accurate Data Acquisition (DAQ) system is proposed in order to calculate the vibration measurements (i.e. eigenfrequencies, damping ratio) of a historical structure in Greece. It was tested in laboratory conditions, where three low-cost triaxial accelerometers (A, B, C) were installed on a steel cantilever beam. They recorded measurements at ambient conditions and under perturbation. A comparative analysis between different software was adopted to test the accuracy and the computational effort of the suggested methodology. Furthermore, a comparison between the results of the suggested DAQ system and that of a commercial system was performed in terms of computational effort and accuracy.

I. System Configuration

A Teensy 4.1 development board microcontroller was adopted to process the collected data and record the signal from the acceleration sensors. The collected data is transferred to a single board PC aiming to process the data and record it if needed. The server used in this investigation is a Rock Pi X B. It is associated with LabView software, where the developed framework is accessible.

The microcontroller sends acceleration time domain data to the server. A limit is set for the acceleration, so that when the sent measurements exceed this limit, LabVIEW starts saving a copy of the results on the server, store it and send a copy to the client via WiFi or Internet Cloud, with a time stamp.

II. Experimental benchmark procedure

Sensors were mounted on the edge of a steel cantilever beam, and measurements were recorded at ambient conditions and under perturbation, by releasing a mass connected to the beam edge. Data from a trademark accelerometer BDI connected to a commercial National Instrument (NI) cDAQ-9135 and the triaxial accelerometers connected to the suggested data acquisition system were recorded.

Fig. 1 Proposed data acquisition (DAQ) system hardware

III. Results

A comparison between MATLAB and LabView software was first performed. Acceleration in time domain was recorded after releasing the mass, where the maximum acceleration reported was +/- 0.25g.

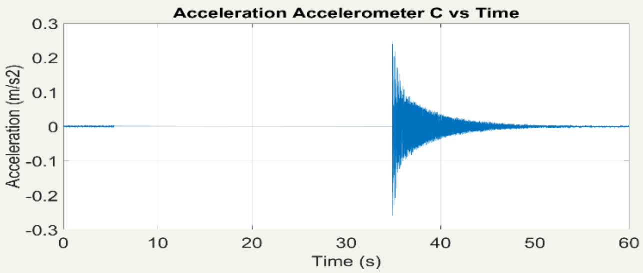


Fig. 2 Acceleration time domain on MATLAB

Fast Fourier Transform was performed on the data of accelerometer C mounted on the steel beam, showing two major peaks at 22.66 Hz and at 30 Hz. For comparison purposes, same data was treated on

LabVIEW. Acceleration time domain showed an acceleration of 0.25g, including noise, but with smoother decrease than the MATLAB output. For the power spectrum at rectangular window, two peaks were detected at 22.66 Hz and at 30Hz.

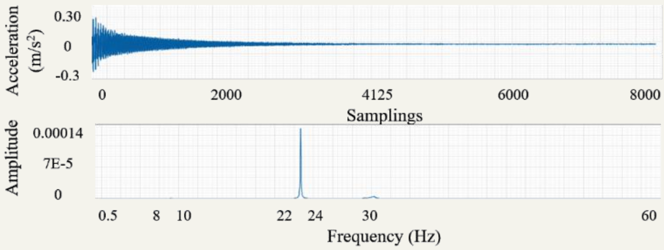


Fig. 3 Acceleration time domain and power spectrum on LabView

Bandwidth Butterworth Filter of order 10 was applied on both software with cut-off frequencies of 20Hz and 30 Hz. Accordingly, noise was removed properly.

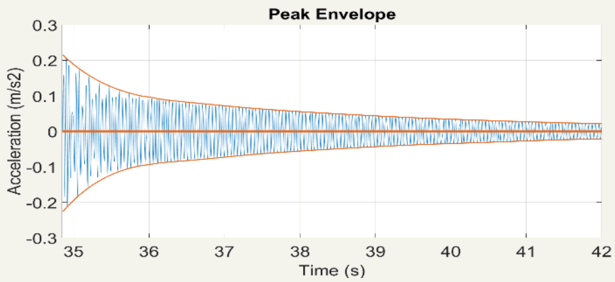


Fig. 4 Peak envelope of filtered acceleration of accelerometer C on MATLAB

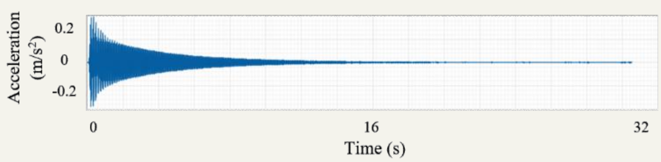


Fig. 5 Filtered acceleration time domain of accelerometer C on LabVIEW

After comparing the results of accelerometer C on both software, one can conclude that both showed identical accurate acceleration before and after filtering. Smoother curves were shown after eliminating noise. Even though same results were calculated, LabVIEW outperformed due to less computational effort.

Then the suggested DAQ system with accelerometer C was compared to the BDI sensor connected to NI cDAQ-9135. The BDI accelerometer recorded a maximum acceleration at +/-0.5g, including noise. The recorded acceleration of the three accelerometers was also accompanied by a lot of noise with a maximum of +0.4g with a sharp decrease.

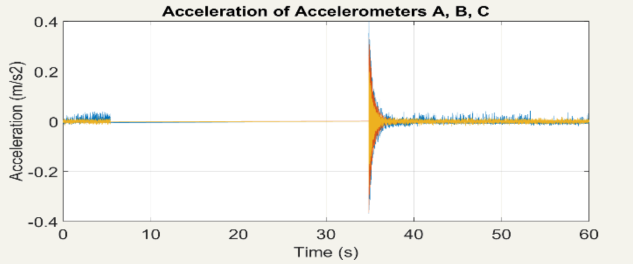


Fig. 6 Acceleration time domain of accelerometers A, B & C in Z direction at perturbation

Acceleration frequency domains were calculated, both showing peaks at 22.6 and 30 Hz. BDI with NI results showed lower noise compared to the results of the three accelerometers.

To compensate for the noise, Bandwidth Butterworth Filter of order 10 was adopted with cutoff frequencies of 25 Hz and 37 Hz. The peak envelops showed a smooth decrease of acceleration recorded for the BDI more than that of accelerometer B.

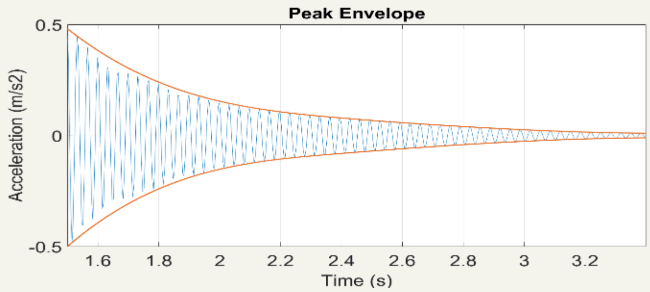


Fig. 7 Peak envelop of filtered acceleration of accelerometer B at Z direction on MATLAB

IV. Conclusion

In conclusion, even though the results do not converge completely, the proposed methodology is sufficient, since the difference is minimal. LabVIEW outperformed MATLAB in terms of computational time. A trade-off between the precision of the outcome and the computational effort should be considered to choose what DAQ system to adopt, but overall, the suggested methodology provided satisfactory results in terms of accuracy, computational effort and network topology considering its limited budget compared to the much more expensive commercial data acquisition system.

by Reina El Dahr, Xenofon Lignos, Ioannis Vayas

Design fault displacement for lifelines at fault crossings: A code approach for Europe

The structural integrity and functionality of lifelines, such as oil, gas, water, and sewage pipelines, as well as elongated infrastructure, such as roads, tunnels, and bridges in the aftermath of an earthquake is decisive for the response management of civil protection authorities and heavily influences the seismic resilience of communities. Among the most catastrophic earthquake-induced actions is the fault offset in the case of large-magnitude earthquakes affecting the overlying structures that have to follow the imposed ground displacement by developing excessive deformation (Fig. 1).

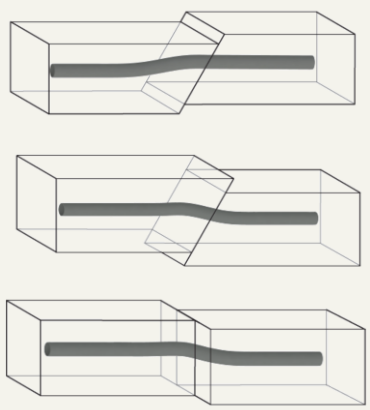


Fig. 1 Fault mechanisms (normal, reverse, and strike-slip) and corresponding deformation of a buried pipeline subjected to faulting

The design fault displacement is typically based on estimates derived from the fault geometry via empirical fault scaling relations for a given "design" scenario event. This approach comes with an unknown level of safety, as it disregards fault activity and the actual distribution of scenarios that it can produce. However, the seismic resilience of critical lifelines and infrastructure should be secured within the framework of Performance-Based Earthquake Engineering. The primary and fundamental step in this direction is the quantification of the fault displacement hazard on the crossing site. The appropriate methodology is the Probabilistic Fault Displacement Hazard Analysis (PFDHA) that aims at quantifying the mean annual frequency (MAF) of exceeding arbitrary fault displacement levels at the lifeline crossing site (Fig. 2), considering the geometrical and seismological properties of the fault together with the location of the crossing lifeline on the fault trace (i.e., the crossing site). However, this is an advanced analysis with complicated probabilistic calculations based on a set of specialized seismological data and thus unsuitable for being incorporated "as is" in code provisions.

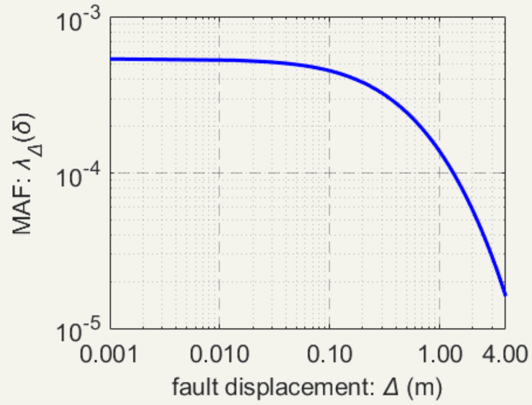


Fig. 2 Illustrative example of a fault displacement hazard curve on the lifeline crossing site

To overcome this problem on a code basis, a simplified approach that allows the (mostly conservative) approximation of the fault displacement corresponding to any given return period based on readily available data, namely fault activity, fault mechanism, fault length, and crossing location was developed. The code-compatible and hazard-consistent statistical approximation was developed for estimating the design fault displacement for European application. A large number of PFDHAs was carried out taking into account the pertinent uncertainties within a logic tree formulation exploiting the seismological and geometrical properties of the database of faults (Fig. 3) considered in the development of the 2020 European Seismic Hazard Model. The methodology comprises a set of equations for calculating the displacement given the fault activity, the fault mechanism, the fault length, and the crossing site.

Fig. 3 Map of seismic faults in the European continent classified per tectonic environment (Interplate: red, Stable Continental Region: blue)

The proposed methodology is implemented as follows:
1st step: The fault mechanism, the fault length, and the crossing point are determined for the lifeline–fault crossing at hand.
2nd step: The activity of the fault is derived either from an available source model, defined by a specialized seismological study or estimated via a proposed approximation.
3rd step: The return period of exceeding a selected fault displacement or vice versa at the lifeline–fault crossing is estimated via a single expression.

The methodology's outcome is illustrated in Fig. 4 for a set of indicative faults located close to industrial areas of low and very high seismicity (Fig. 4).

The proposed methodology has been adopted as an informative Annex in the 2020 version of EN1998-4 and it may serve as a screening tool for lifeline route selection, or even as a preliminary design tool to indicate when a more specialized study is needed.

Associate Professor Dimitrios Vamvatsikos is a member of the Technical Committee CEN/TEC250/SC8 for Eurocode 8 – Part 4: Silos, tanks and pipelines, towers, masts and chimneys.

by Vasileios E. Melissianos and Dimitrios Vamvatsikos

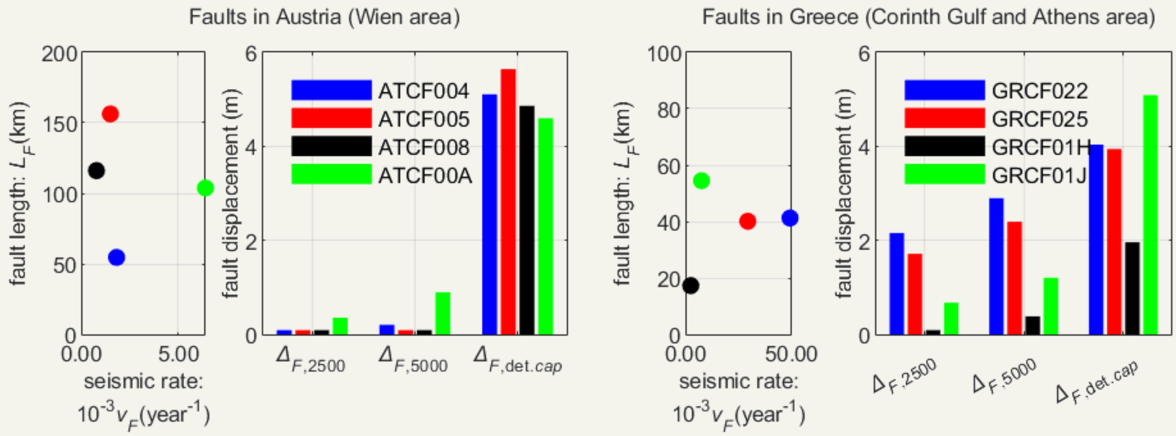


Fig. 4 Comparison of fault displacement obtained from the EN1998-4 approach for return periods of 2500 years and 5000 years, also showing the deterministic cap (displacement obtained via empirical fault scaling relations) for faults in Austria (low seismicity area) and Greece (very high seismicity area)

PhD Defense of Marios-Zois Bezas

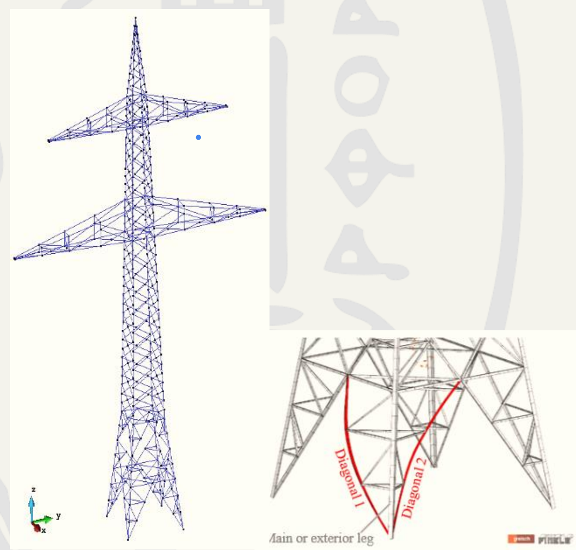
In October 2021, Marios-Zois Bezas successfully defended his PhD dissertation entitled “**Design of lattice towers from hot-rolled equal leg steel angles**”.

Angles profiles have been used since the very beginning of steel construction due to their easy transportation and on-site erection. However, they exhibit specific features that clearly distinguish them from other types of common sections, what inevitably leads to the need for the development of specific design provisions.

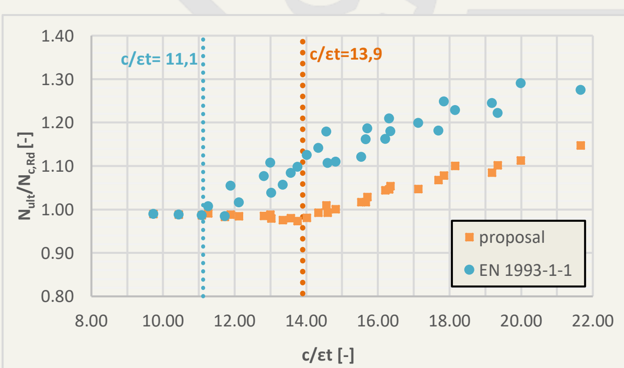
In a first step, existing European specifications on hot-rolled equal angle sections were critically reviewed and then, in a second step, extensive experimental, analytical, and numerical studies have been conducted to propose a complete and duly validated set of design rules covering all aspects of design for angles. These rules include cross section classification, cross section resistance for all types of loading as well as rules for member design to individual and combined internal normal forces and moments. All the proposed rules are written in Eurocode 3 format to allow a direct possible inclusion in forthcoming drafts.

Furthermore, angle profiles are extensively used in lattice towers and masts for telecommunication purposes or electric power transmission. Such types of towers are mainly designed according to EN 1993-3-1 and EN 50341-1, based on a first-order linear elastic structural analysis of a truss structure. An assessment of the current design approach is performed, where the tower is simulated with a full non-linear finite element software, considering relevant imperfections as well as geometrical and material non-linearities. The importance of the second order effects in the analysis is underlined while the existence of an instability mode not properly covered directly by the norms, and usually therefore not checked, is highlighted. Two analytical models for the prediction of the critical load of this buckling mode are proposed and validated numerically. Both proposed models are rather easy to apply and may fill the gap in the existing design recommendations for lattice towers.

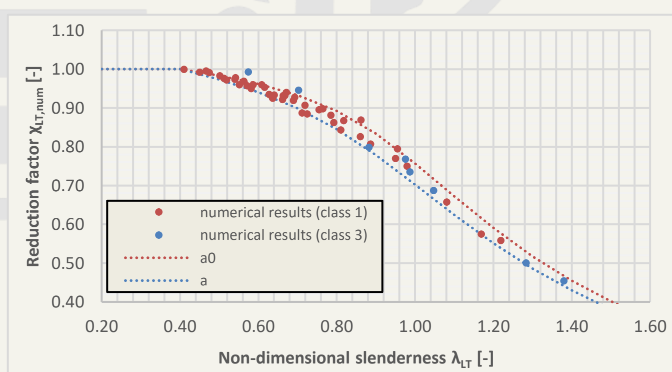
Compression tests on large angle HSS



3D finite element model for lattice tower (left) and segment instability mode (right)



Ratio between numerical results and design resistance to compression obtained from the current proposal and EC3



Comparison of numerical results with buckling curves for LTB of EC3

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