

Phaedon Karydakis,
Massimo Majowiecki

The Olympic Airways Hangar at the new Athens International Airport

REALIZZAZIONI

The new Olympic Airways Hangar consists of five sections: the office and maintenance supporting multistoried buildings (sections 1, 2 and 3) and the two hangar bays (sections 4 and 5) with main plan view dimensions of 85.0*150.0 meters each, as indicated in the key plan of fig.1.

The conceptual hangar framing system of the original design, presented as a guideline in the competition, adopted a structural typology more often used for small and medium size hangars (fig.1), corresponding to the following description:

1. Roof deck is 40mm x 150mm metal deck with screwed fasteners. The roof deck will provide a diaphragm that will transfer lateral loads at the roof level to the structural steel frame.
2. Roof framing consists of the steel joints or hot-rolled purlins spanning from spandrel beams to trusses or from truss to truss.
3. Two lateral-load resistant, 12m deep portal trusses are used to span the opening at the hangar doors.
4. Variable depth (maximum depth = 8 m) simple-span trusses span from the columns along the back wall of the hangar to the portal trusses.
5. All roof trusses are fabricated from hot-rolled, wide-flange, structural steel shapes in a weak-axis orientation with bolted gusset plate connections.
6. An horizontal truss at the bottom chord of the roof trusses (horizontal bracing level) is used to transfer lateral forces into the hangar sidewalls.

The new Olympic Airways hangar 300 m long and 85 m wide with 25 m usefull internal high is actually under a final phase of construction. A space frame structural system has been adopted to realize 5 double trusses longitudinally oriented supported laterally and in the transversal axis of simmetry. The weight of the steel structures is approximately 80.000 kN of open profiles and composite sections obtained by automatic welding of plates. The erection system was provided by two special cranes able to lift up 11.000 kN on the top of the supporting transversal frame. By sliding on Teflon pads, the composite space frame was positioned on the related restrain details.

L'hangar delle Olympic Airways del nuovo Aeroporto Internazionale di Atene

Il nuovo hangar delle Olympic Airways attualmente nella fase finale di costruzione è lungo 300 m, largo 85 m ed ha un'altezza interna utile di 25 m. Per la sua realizzazione è stato utilizzato un sistema strutturale spaziale a telaio per realizzare 5 doppi tralicci orientati longitudinalmente con supporti sia laterali sia nell'asse trasversale simmetrico. Per la sua realizzazione sono stati impiegati c.a 80.000 kN di strutture metalliche (profili aperti e sezioni composte ottenuti da saldatura automatica delle lamiere). Per il montaggio sono state utilizzate due gru speciali in grado di sollevare 11.000 kN sulla sommità della struttura portante trasversale. La struttura spaziale composta è stata posizionata nelle rispettive sedi di vincolo movimentandola su cuscinetti in Teflon.

7. Hangar sidewalls are X-brace to transfer lateral forces from the roof diaphragm into the foundation system.

The wide span unobstructed area, required for the

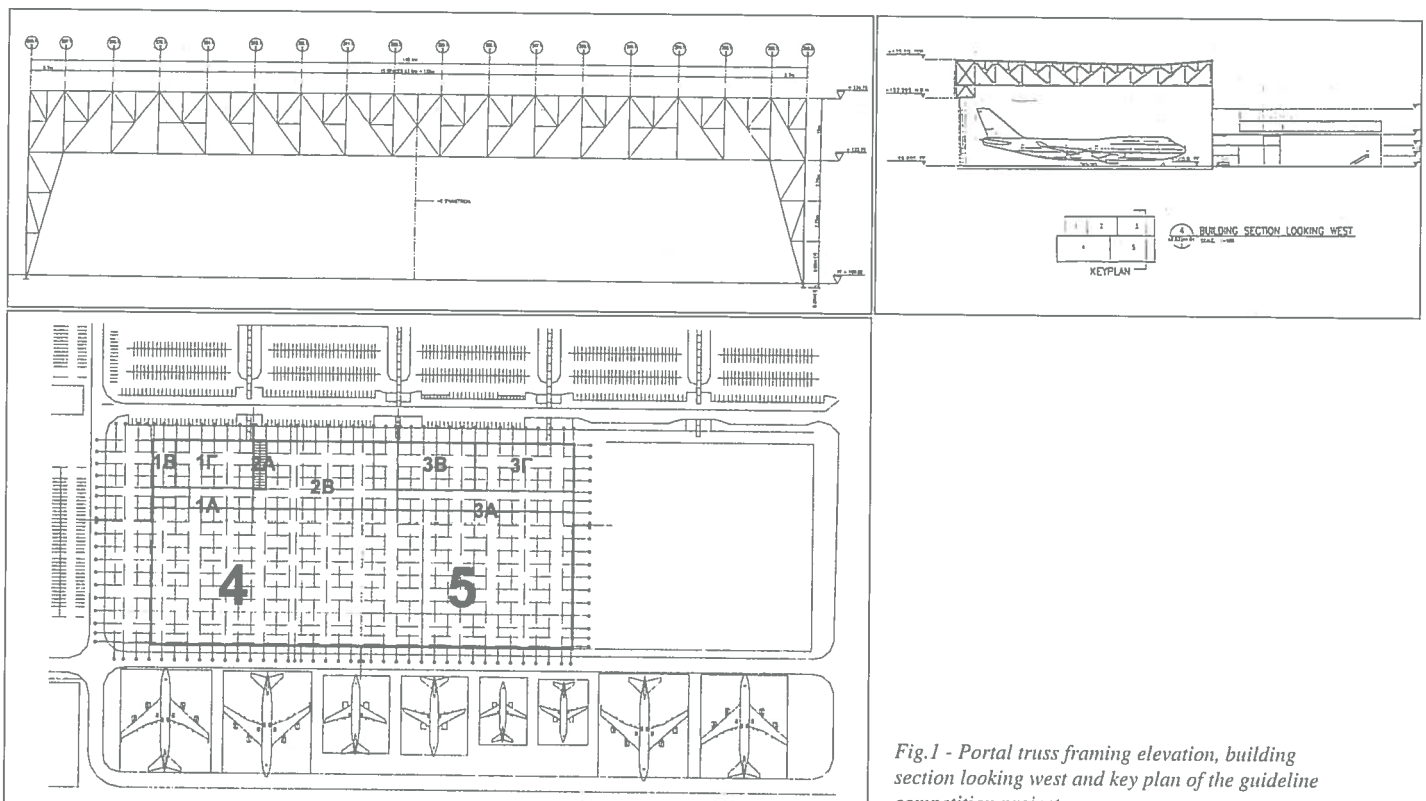


Fig.1 - Portal truss framing elevation, building section looking west and key plan of the guideline competition project.

functionality of the hangar, called for a 2x150m long span structural system and, therefore, the structural system of the two bays, initially independent, was redesigned in order to increase the reliability of a long span structure considering the high seismicity of the site.

The conceptual alternative design solution was based principally on the necessity to obtain a clear distribution of the seismic forces and, at the same time, to avoid high thermal coactive stresses. In fact, the hyperstatic two hinged frame, generating high horizontal forces at the base, creates a parametric sensitive structural response due to the different level of reliability of the foundation system; the horizontal displacement of a pile foundation group, under a strong motion seismic action, introduces an unacceptable variation of the stress and displacement states of the frame. On the other hand, a relatively more efficient solution, adopting an underground prestressed tie system, was considered uneconomical.

The structural solution presented in this paper (fig.2), won the International competition also by saving steel consumption and giving many other functional benefits, illustrated in the following points.

DESCRIPTION OF THE STRUCTURAL SYSTEM

Buildings 1-2-3

Buildings 1, 2 and 3, are mainly concrete buildings, with flat concrete slabs, supported on composite columns of steel H sections encased in concrete, with a basic column grid of 9.0*9.0 meters.

The structural system of these multistoried (1-4 levels) buildings consists of reinforced concrete flat slabs, with a thickness of 30 cm, supported without beams on composite columns, formed from steel H sections, encased in concrete. The collaboration of the two materials is assisted by shear welded studs. On the column head, special antipunching reinforcement (DEHA) is used, even if critical factor in the design of the slabs is the deflection due to the flexibility and creep, and not the flexural or punching strength.

All earthquake forces are sustained by shears walls, evenly distributed in all sections, the composite columns and the flat slab connection were assumed pin jointed.

The shear walls are positioned in such a way to withstand earthquake loads without adding significant loads from temperature variation, creep and shrinkage.

Buildings 1, 2 and 3 are designed with the minimum amount of expansion joints for maximum serviceability.

Buildings 4-5

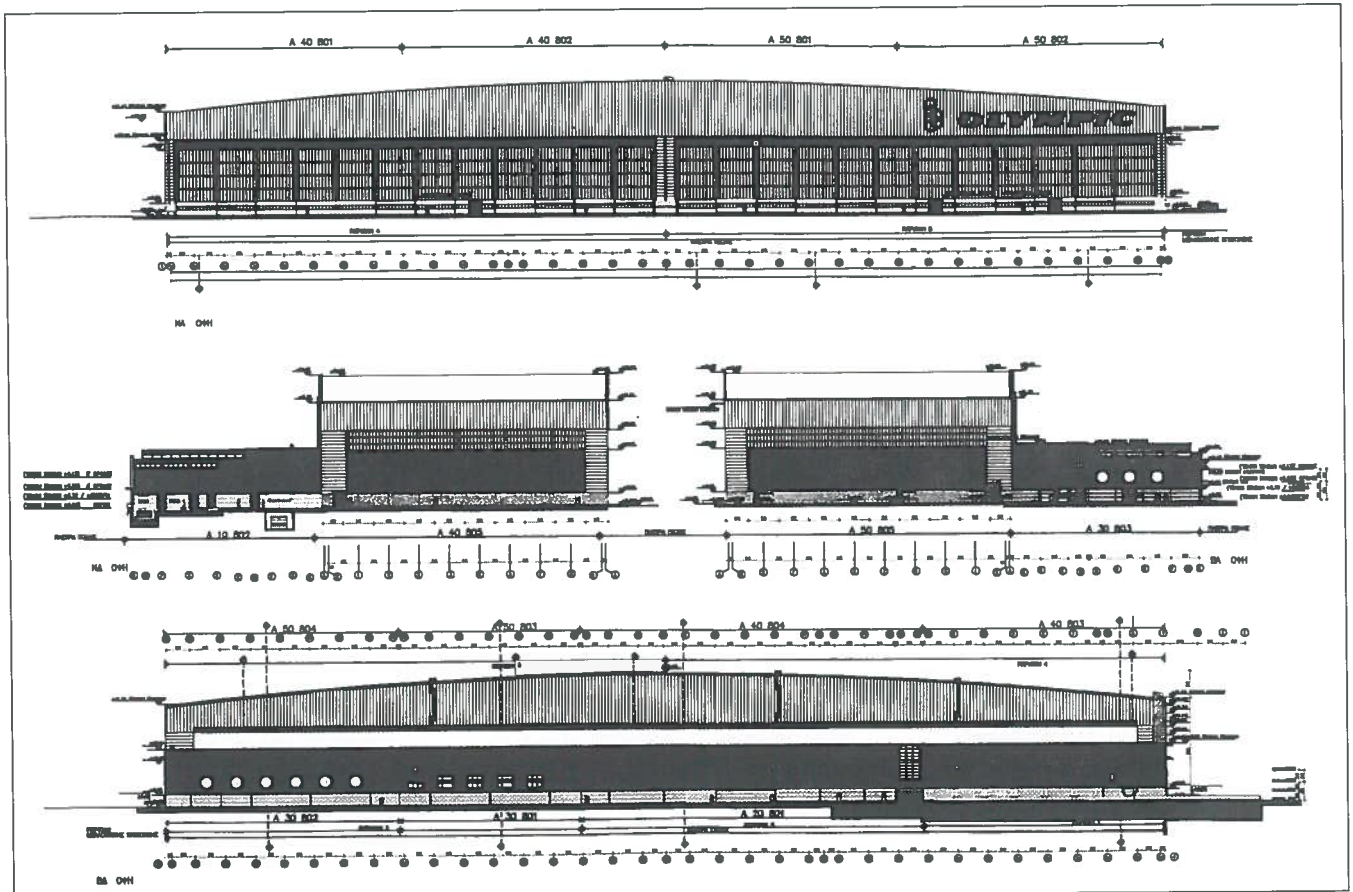
Buildings 4 and 5, principal object of the present paper, are the two long span bays of the Hangar.

The structural system of the hangar is mainly formed by:

- ten parallel twin trusses longitudinally oriented;
- three trasversal composite portal frames;
- a suspended system of external walls;
- a roof deck covering system.

The parallel 300 m long twin trusses, longitudinally oriented, supported on three main transversal box girders, are positioned with a relative separation of 9.115 m. The trusses have a curved top flange.

Fig.2 - Front view of the hangar.



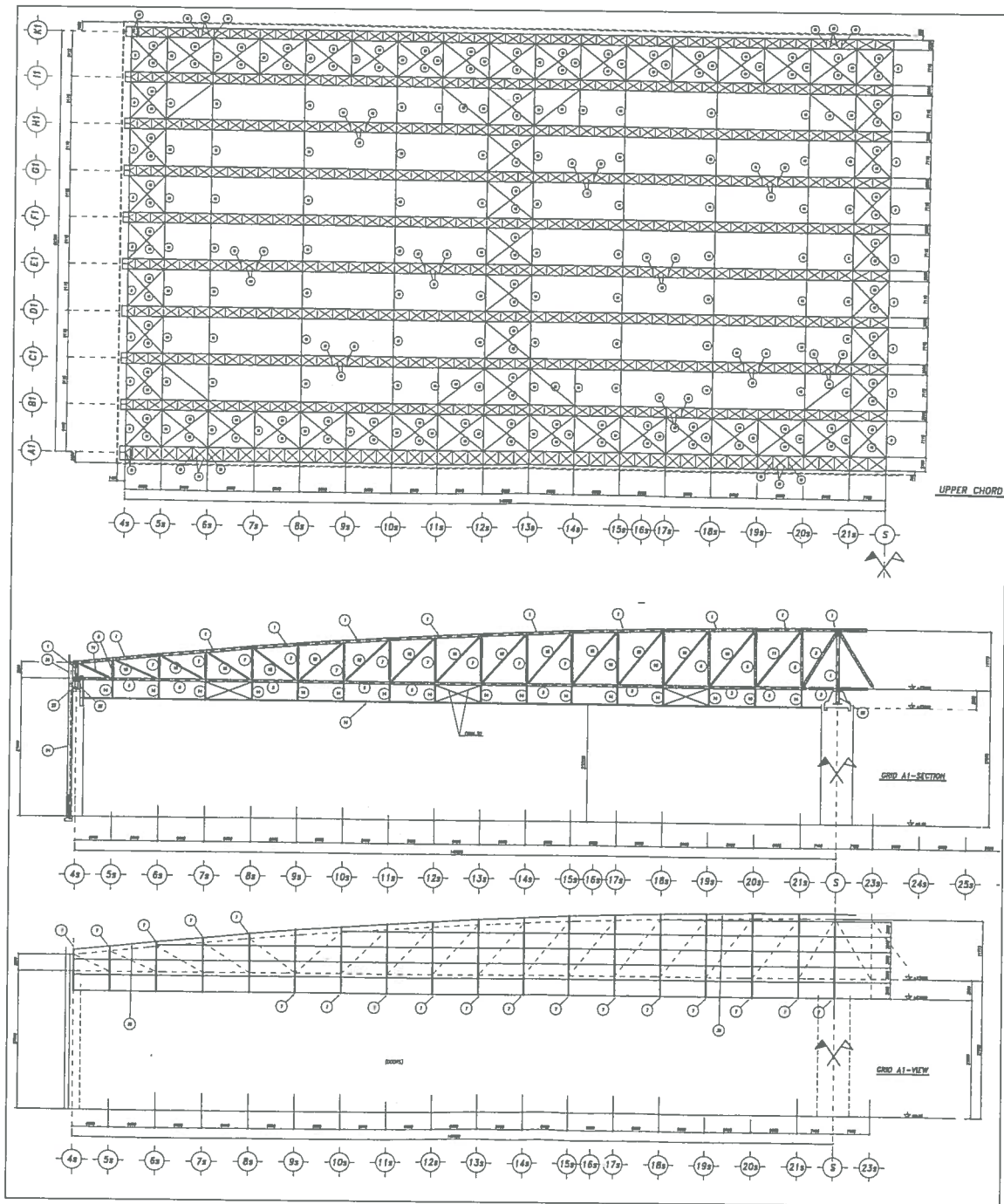


Fig.3 - Plan view and elevation of the structural system.

ge, with a height varying from 3.5 meters at the ends to 12.5 meters at the center, and a bottom horizontal flange (fig.3). The upper chord geometry facilitates the rain drainage in comparison with the tender design.

Two couples of twin trusses are then both connected, with vertical and horizontal bracings, in order to finally obtain a self out of plane stabilizing space frame, forming the principal structural system (Fig.4).

The five boxed space frames, are supported on transversal oriented frames formed by 4 box-section concrete columns and a steel box-section continuous beams, with three spans 27.5 meters each (85 meters long), positioned along the two lateral gable walls and the axis of symmetry of the Hangar.

Because of their principal structural role and critical cantilever action, the central columns are desi-

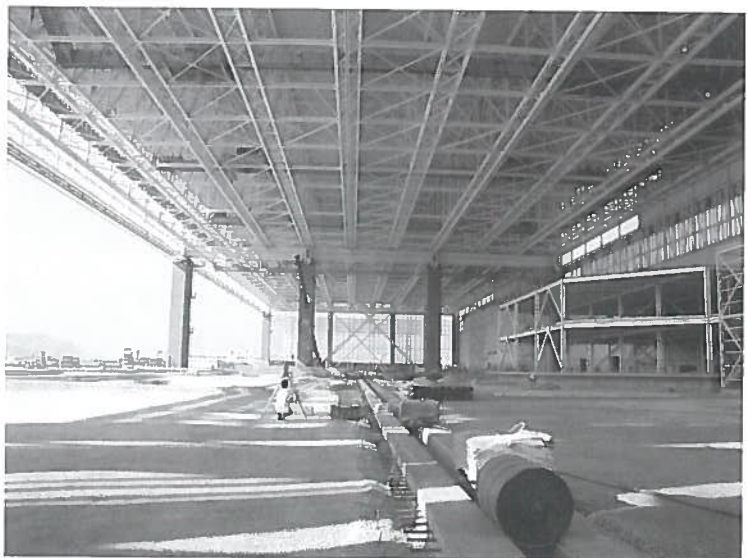


Fig. 4 - Assembly and erection of the main steel box girders.

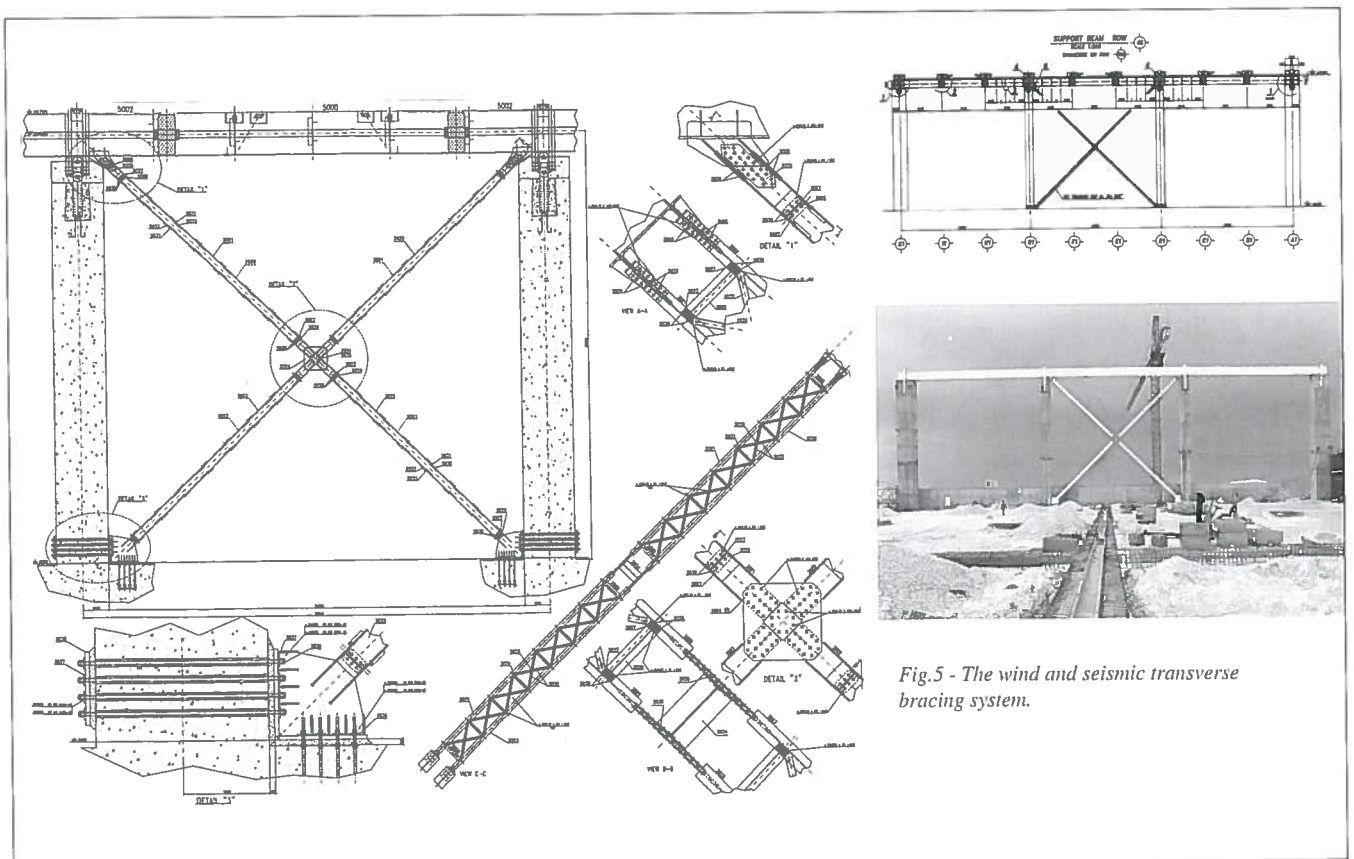
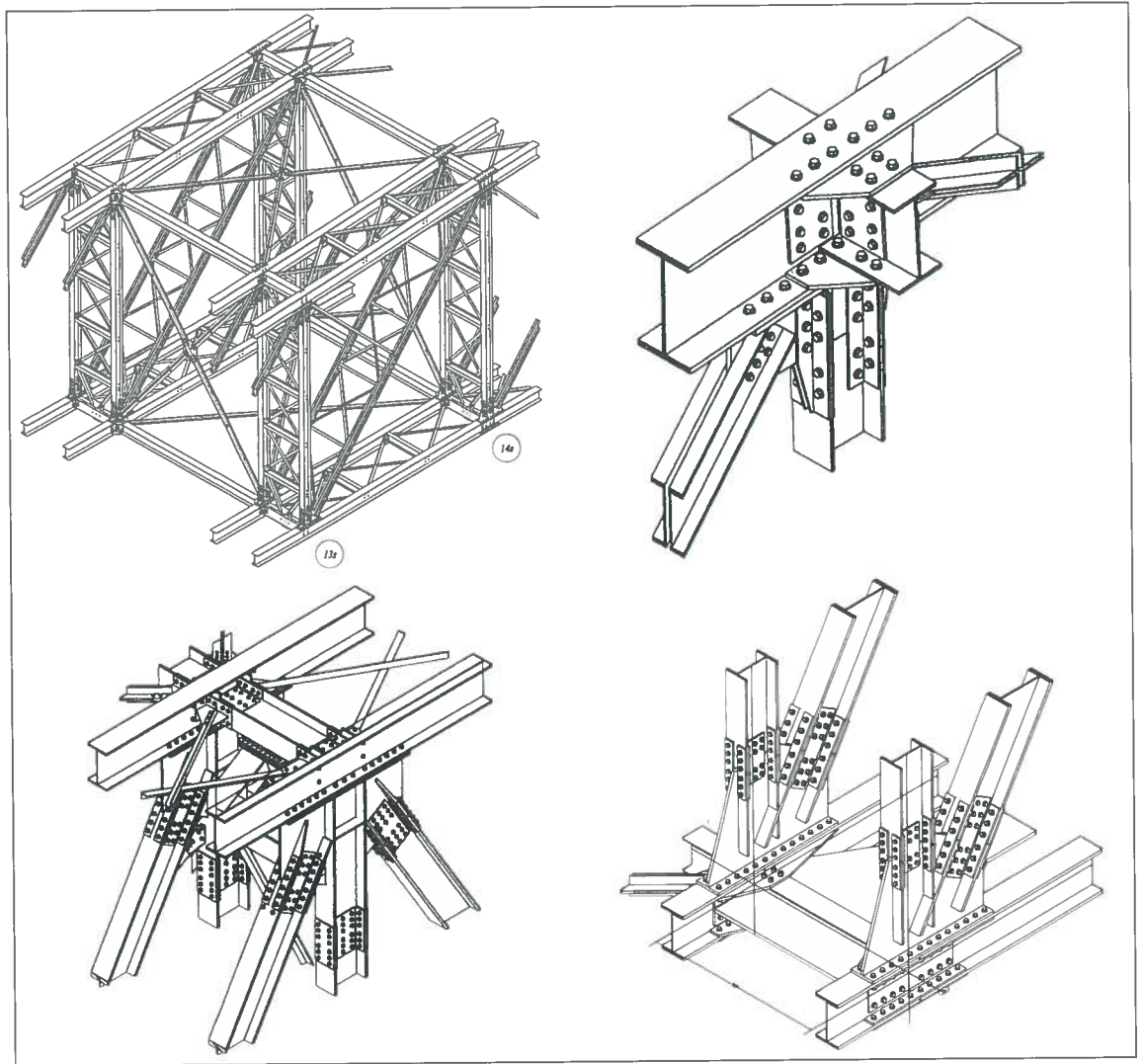


Fig.5 - The wind and seismic transverse bracing system.

igned as very strong hollow box sections (6.0*3.0 meters) founded on concrete piles.

In the transverse direction, seismic and wind forces are resisted by steel diagonal cross bracing composite sections placed between the central pairs of concrete columns of the laterals and central transversal frames forming, globally, a seismic, ductile controlled, vertical wind bracing system (Fig.5).

All singular twin trusses are supported on the box girder beams, through elastometallic bearings, in the same manner as the box girders are supported on the concrete columns. All bearings have spherical rotation capability, while the horizontal movements are limited depending on the position of the bearing. The bearings along the axis of symmetry are fixed, while the bearings at both ends allow for horizontal movement, along the main axis, for thermal expansion.

With these restraint conditions, the central columns on the axis of symmetry will resist all seismic and wind forces along the longitudinal axis, with a cantilever action, while the columns at the ends will undertake seismic load with use of seismic buffers in the event of exceptionally large horizontal displacements (fig.6, 7). Regarding the lateral and gable wall, a suspended system is adopted in order to minimize foundation construction, increase the stability properties of a high wall slender structure and give an easy system for future lateral enlargement of the hangar.

The two 85m long and 30m high walls are directly suspended from the heads of the main trusses following the same structural geometry of twin profiles interconnected with a Vierendell typical transversal connection ; the main back wall is suspended from the corresponding trusses, the slotted lower connection with the maintenance multistory building gives the possibility of longitudinal thermal expansion and vertical differential sliding displacement .

The front door, 22.70m high, involves the first long span trusses to carry the track lines for the longitudinal retractable door system. All the trusses supports an inner system of platforms and catwalks addressed to the functionality of the hangar.

The final covering deck is done with trapezoidal corrugated galvanized and prepainted steel sheets, additional insulating material and a waterproofing metallic layer . Since the two parts of the twin trusses are 2.0 meters separated and the axis of each truss is 9.115 meters apart, the trapezoidal sheet

Fig.6 - Detail connection of the central concrete columns and the transversal box beam - restrained in x and fy direction

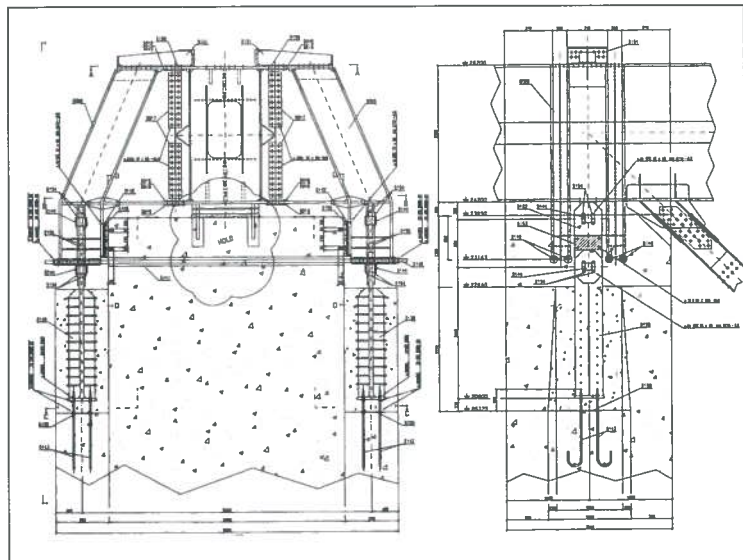
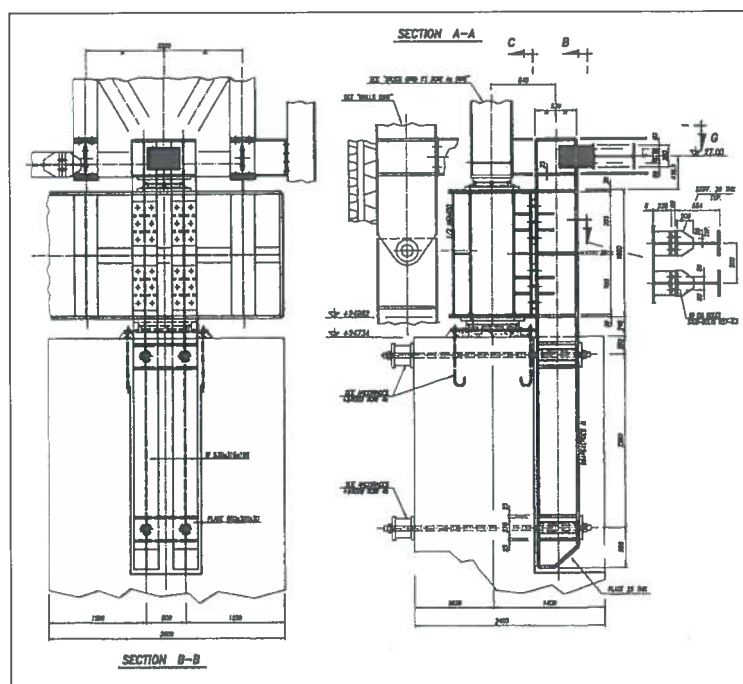


Fig.7 - Detail connection of the lateral concrete columns and the transversal box beam - restrained in y direction.



spans a maximum of 7.115 meters avoiding the use of additional, transversal oriented, purlins.

MATERIAL DESIGN DATA

Concrete-normal weight (minimum ultimate compressive strenght of cylinders at 28 days, tested in compliance with ENV 206)

- Foundations (C25): 25 MPa.
- Support area slab on grade (C30): 30MPa.
- Hangar slab on grade including trench (C35): 35 MPa
- Precaste concrete panels (C35): 35 MPa.
- All other concrete, unless noted otherwise (C30): 30 MPa.

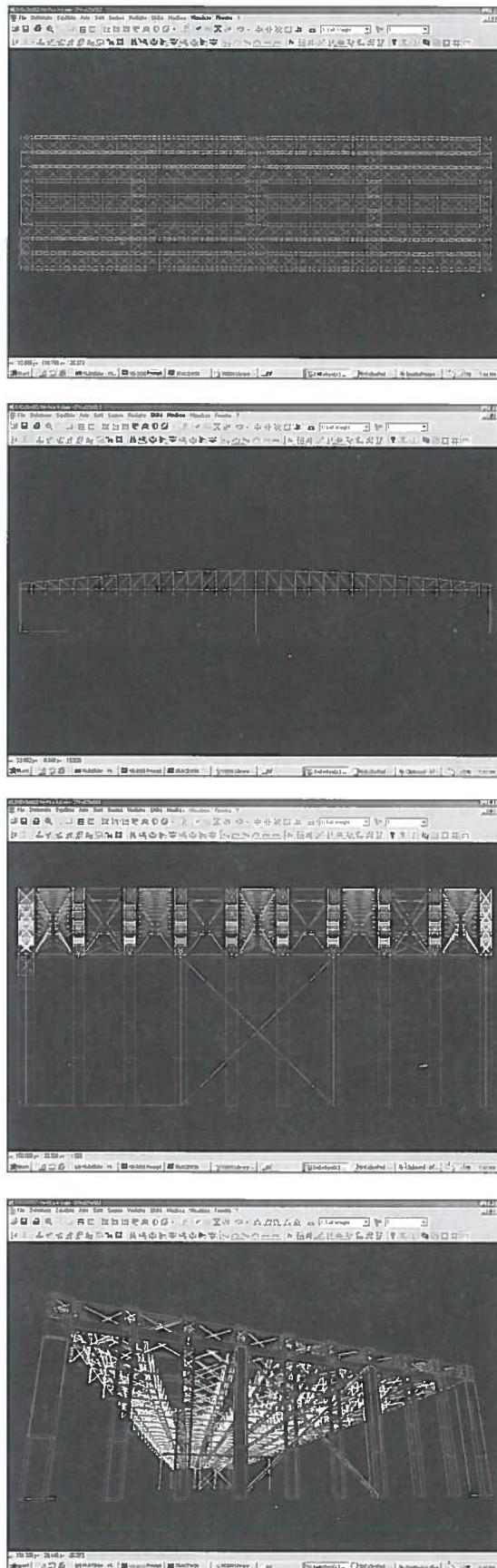
Concrete reinforcement (minimum yield strenght, per EMV 10080

- All deformed bars (S500): 500 Mpa, Class S.
- Welded wire fabric (S500): 500 Mpa, Class S.

Structural steel (minimum yield strength, FY)

- All hangar wide flange columns, wide flange struts, vertical bracing wide flange shapes, horizontal bracing truss diagonals and struts, bracing truss wide flange chords, and all shapes in trusses (EN 10025 Grade FE E 355): 355 MPa.
- All other hot-rolled shapes, double angles, an-

Fig.8 - The mathematical model of the hangar.



gles, plates and anchor bolts (EN 10025 Grade FE E 235): 235 Mpa.

GOVERNING CODES & STANDARDS

Vertical loads, wind loads, thermal loads and loading combinations

- Eurocode No. 1 - "Basis of design and actions on structures"
- DIN 1055-1 - "Design loads for buildings"

Seismic design

- Eurocode No. 8 - "Design provisions for earthquake resistance of structures"
- "New Greek Seismic Code" (NEAK) (Government Gazette, 534B,588B, 1995)

Geotechnical aspects of foundation design: (Eurocode No.7 - "Geotechnical Design"

Concrete

- Eurocode No. 2 - "Design of concrete structures"
- "New Greek Code of concrete construction works" (Government Gazette 227B, 1995)
- "Greek Code of concrete technology"
- ENV 206 - "Concrete - Performance, placing and compliance criteria"
- ENV 10080 - "Steel for reinforcement of concrete"
- DIN 1045 - 1 - "Structural use of concrete"
- DIN 4226 - 1 - "Aggregates for concrete"

Clay tile brick and concrete masonry units

- Eurocode No. 6 - "Design of masonry structures"
- "New Greek Seismic Code" (NEAK) (Government Gazette, 534B,588B, 1995)

Structural steel: Eurocode No. 3 - "Design of steel structures"

- Open-web steel joists and joist girders: Eurocode No. 3 - "Design of steel structures"
- Composite floor systems: Eurocode No. 4 - "Design of composite steel and concrete structures"
- Metal deck: Eurocode No. 3, Part 1-3 - "Cold formed thin gauge members and sheeting"

THE MATHEMATICAL MODELS

For the analysis of the main structure for buildings 4 and 5 , a full space model for the whole structure was used.

The model included the trusses, the bracings, the main girders, the concrete columns, and the main diagonal aseismic bracings and the bearings.

The model shown in fig.8, was analysed using the specialized analysis software: WINSTRAND - 2001 by ENEXSYS.

The analysis and dimensioning of the structure was done for the most critical combinations, taking into account non linear geometrical and material analysis when necessary.

The concrete columns are designed according to Heft 220 DTAFSTB and DIN 1045, taking into account all permanent, live and accidental loads, initial eccentricity and deformation due to creep from the actions of permanent loads

Loading analysis and deflection criteria

- Roof snow load: 1 kN/m²
- Wind load - Basic wind speed: $V_{ref} = 30$ m/sec
- Thermal load:
 1. Minimum temperature = +40 °C / -30 °C
 2. Minimum service temperature = 0 °C
- Deflection criteria:
 1. Crane systems: crane live load deflection shall not exceed 1/1000 the runway span
 2. Hangar roof framing: camber roof trusses to offset anticipated dead load deflection. Maximum door truss deflection: 25 cm to be coordinated with hangar door manufacturer
 3. Exterior wall girts shall be designed to meet composite metal wall
- Panel service ability criteria as follows:
 1. Maximum inward or outward deflection $< L/250 < 20$ mm max
 2. Maximum vertical deflection $< L/750 < 12$ mm max
 3. Erection tolerance of (0.1%) shall be maintained during erection.
- Load Conditions (see Table 1)
The calculation code perform a non-linear analysis iterating across the load case for unilateral tension stress in horizontal wind bracings.
- Load Combinations (see Table 2).

In the previous load case the following rules are considered:

- In the load combination 3 & 4 the imperfections forces are increased by a factor 1.56 in order to take into account the increase of average magnitude of normal forces in the chords member of main beams.
- The load case 5 (Crane) is incremented by a factor 1.5.
- When Wind forces are considered, according to EC3 and DAN implementation rules, no imperfections need to be considered.
- When Wind forces are considered, in order to maximize the uplift effect, self weight, dead load, and crane effect are factorized by 1.

No thermal variations are included considering the fact that, thanks to global restrain, no force increments are induced by this load case.

Concerning the structural response and verifications, it has been considered interesting to report, in the following, some special problems encountered during the design process, avoiding to extend the

paper with conventional diagrams.

The vertical wind and seismic bracing system

The transversal stability of the hangar is given by a macro X-bracing system placed in the central bays of the portal frames.

According to the Greek seismic code and the recommendations of the Eurocode n°8, the bracing system and the steel structure has been designed in order to withstand the seismic actions with a q-factor =3.

Under this assumption the members and connections,involved in the verifications, have to satisfy the following limitations:

- In tension element jointed with bolts: $A_{net}/A > 1.262 f_y/f_u$
- Bolted joints with fillet welds: $R_d > 1.2 * R_{ty}$ where R_d = the ultimate strenght of the connection and R_{ty} = the ultimate strenght of the jointed member
- In vertical and horizontal bracing (aseismic bracing) the members must withstand design forces with seismic combinations increased by a factor of 1.5
- The non-dimensional slenderness of vertical aseismic cross-bracing even when only the tension member is calculated to resist the seismic forces, should not exceed $\lambda = \sqrt{A * (f_y/N_{cr})} \leq 1.5$

Load Case	Comment
1	Self Weight
2	Dead Load
3	1 kN/m ² Left
4	1 kN/m ² Right
5	Crane Left
6	Crane Right
7	Wind blowing from Y+ Doors Open
8	Wind blowing from Y+ Doors Close
9	Wind blowing from Y- Doors Open
10	Wind blowing from Y- Doors Close
11	Wind blowing from X+ Doors Open
12	Wind blowing from X+ Doors Close
13	Seismic Forces on X direction
14	Seismic Forces on Y direction
15	Imperfections on X- Y-
16	Imperfections on X- Y+
17	Imperfections X+ Y+
18	Imperfections X+ Y-
19	Doors Railway

Table 1

Table 2

Load Combinations	Load Case																		
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1 SelfWeight	1.35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2 Dead Load	1.35	1.35	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1.35
3 Snow	1.35	1.35	1.5	1.5	1.4	0	0	0	0	0	0	0	0	0	1.56	1.56	1.56	1.56	1.35
4 Snow Asym	1.35	1.35	1.5	0.75	1.4	0	0	0	0	0	0	0	0	0	1.56	1.56	1.56	1.56	1.35
5 Wind Y+ Open	1	1	0	0	1	0	1.5	0	0	0	0	0	0	0	0	0	0	0	1.35
6 Wind Y+ Close	1	1	0	0	1	0	0	1.5	0	0	0	0	0	0	0	0	0	0	1.35
7 Wind Y- Open	1	1	0	0	1	0	0	0	1.5	0	0	0	0	0	0	0	0	0	1.35
8 Wind Y- Close	1	1	0	0	1	0	0	0	0	1.5	0	0	0	0	0	0	0	0	1.35
9 Wind X+ Open	1	1	0	0	1	0	0	0	0	0	1.5	0	0	0	0	0	0	0	1.35
10 Wind X+ Close	1	1	0	0	1	0	0	0	0	0	0	1.5	0	0	0	0	0	0	1.35
11 Seismic X	1	1	0.5	0.5	1	0	0	0	0	0	0	0	1	0.33	0	0	0	0	1
12 Seismic Y	1	1	0.5	0.5	1	0	0	0	0	0	0	0	0.33	1	0	0	0	0	1

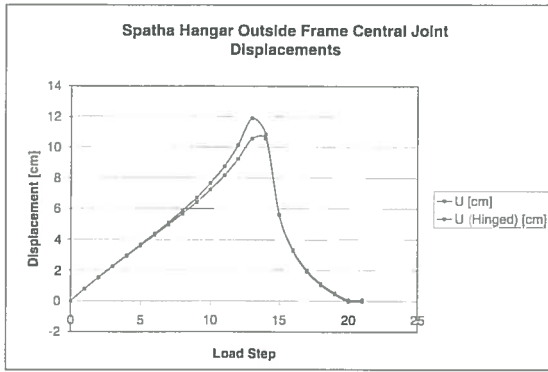
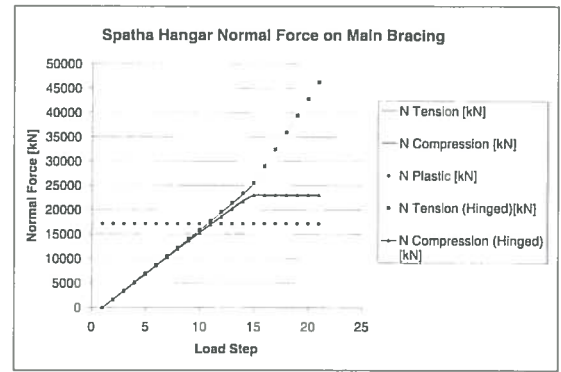


Fig. 9 - Load vs horizontal out of plane displacement.

Fig.10 - Axial tension bracing force vs load increment.



At the same time, for ductility requirements, the connection details must have a local overstrength in respect of the tension ductile member $R_d > 1.2 R_{fy}$. Concerning the bracing members constructed by a composite section of H profiles a preliminary verification have been done according to DIN 4100, giving the possibility to compute the equivalent slenderness of the member under compression, considering a buckling length between 0.5/0.6 of the global length.

A more precise analysis, considering the stabilization energy of the tension member has been made by a second order incremental non linear analysis, in order to find confirmation of the previous results. In fig. 9 and 10 are illustrated the load vs horizontal displacement of the X-frame showing the non linear behaviour of the bracing before collapse and the axial bracing tension force vs load, respectively. The central crossing joint of the bracing system start with an out of plane displacement of 1/500 of the member length; it is interesting to observe that the joint have an initial out of plane increment of displacement but, when the geometric stiffness of the tension member start to govern (hardening behaviour) the joint start to return to the initial position stabilizing the system.

The critical loading factor of the wind bracing system resulted to be higher of the plastic axial force of the bracing member; the slenderness of the compression member did not affect the ultimate capacity of the system, assuring the safety under the requested ductility conditions against the seismic actions.

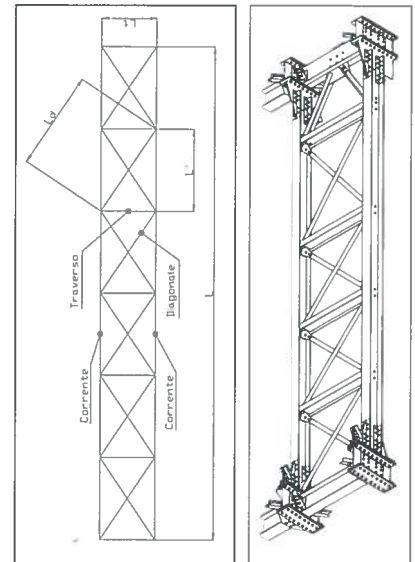
On the composite section design and ULS stability verification of the elementary members of the double truss system

Another interesting item during the design conception of hangar was the choice , under practical and economical boundary conditions, of the composite trasversal sections of the upper,bottom, posts and diagonals of the double truss system.

The composite standard section is mainly due by the necessary stability under transitory erection procedure variation of state of the structure. The length of the single members gave, under nominal stress verification, a very high slenderness factor therefore a twin separated profiles permitted to obtain a well balanced slender factor in and out of plane of the inertial mechanical characteristics of the composite structural beam member.

A parametric evaluation of the separation distance between the horizontal transversal posts and the metallic area of diagonal considering the following

Composite braced post and diagonals.



main functions of the bracing pattern:

- 1) The absorption of the horizontal action generated by outside loads, such as the wind, seismic action, and so on, and their transfer to the vertical counterbraces (resistance function);
- 2) By identifying a suitably rigid reticular pattern, prevent the overall instability of the compressed current of the main structures subject to the action of the main vertical loads (stiffness function).

If we leave aside the first function, with respect to which the dimensions of the elements of the counterbracing (diagonals and transverse beams) takes place in line with classical criteria (determination of the efforts induced in the shafts of the resistant reticular pattern adopted by the outside loads considered, followed by the dimensioning of the profiles), the second function may be achieved by means of the criterion set out here, based on the definition of the minimum axial stiffness necessary for the counterbracing diagonals.

The method

Let us consider the pattern set out below.

This is the plane view of a counterbracing system made up of two currents connected to each other by crossbeams and diagonals. In the resistance to the main vertical loads, the currents are the main structure of span L. In relation to span L, this structure is normally set up using full core beams mainly subject to flexion, or reticular beams.

In both cases, we have the problem of guaranteeing

the stability of the compressed part of the main structure (upper wing of the full core beam or upper current of the reticular beam) at the level of the layer.

In the hypothesis – favourable to safety – that the axial compression force along the currents is constant, the expression of the critical load on the compressed current is:

$$N_{cr,eu} = \pi^2 EJ_y / L^2$$

in accordance with the classical Euler formulation, or

$$N_{cr} = A \sigma_{crit} (\lambda / \lambda_c)$$

where

E = elastic module of the material;

J_y = moment of inertia of the compressed wing or the compressed current with reference to the unstable plane considered;

A = area of the compressed wing or current

$\sigma_{crit} (\lambda / \lambda_c)$ = Critical tension expressed in accordance with the ratio between the slenderness λ of the compressed element ($\lambda = L/i_y$) and the parameter $\lambda_c = \pi \sqrt{E / f_y}$ which depends on the material (= slenderness corresponding with the limit of validity of the behaviour in the purely elastic phase of the shaft). In this context, see point 7.2.2.3.2 of CNR-UNI 10011.

In accordance with the trellis system adopted, the geometric characteristics of the trellises and the static characteristics of the shafts making it up, it is possible to calculate the critical load or equivalent slenderness λ_{eq} of the shaft made up in this way, and consequently the equivalent ratio $(\sigma_{crit} / f_y)_{eq}$.

After establishing all the geometrical and static parameters of the trellis pattern adopted, with the exception of the area A_d of the profile used as a trellis diagonal, and by varying this, it is possible to calculate and draw up the graph of the curve $(\sigma_{crit} / f_y)_{eq}$ in accordance with A_d .

As we can imagine, this curve grows as parameter A_d increases, in other words, as the diagonal area increases, the phenomenon of instability of the compressed current of span L is reduced.

If we consider the instability of the single portion of compressed wing or current on local span L' , we obtain:

$$\lambda_{loc} = L' / i_y \text{ from which we obtain } (\sigma_{crit} / f_y)_{locale}$$

If we enter this value in the previous graph (constant as A_d varies, as it does not depend on this) we can indicate with $A_{d,ott}$ the value of A_d where the two curves meet, obtaining $(\sigma_{crit} / f_y)_{locale} = (\sigma_{crit} / f_y)_{eq}$.

The value $A_{d,ott}$ is the maximum effective value of the area of the diagonal with a view to maintaining the stability of the compressed current as, with greater values, the prevalent phenomenon that takes place for the lesser N_{crit} is local instability of the current at span L' .

The graph in fig. 12 shows the curves (in accordance with A_d):

1) of local slenderness λ_{loc} ;

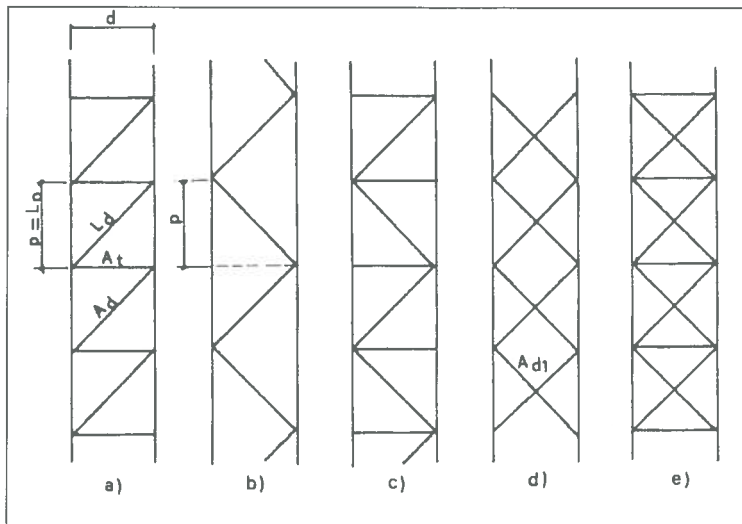


Fig. 11 - Figure taken from "Steel Structures", Ballio-Mazzolani.

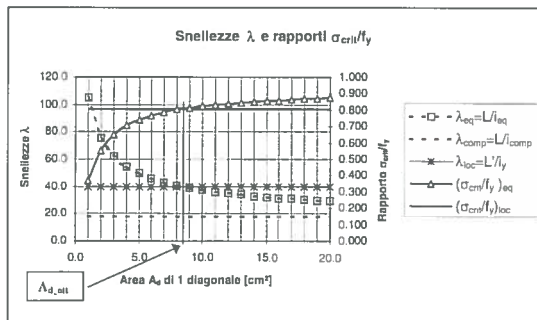


Fig. 12 - Optimal choice of diagonal bracing steel section of composite member vs slenderness ratio.

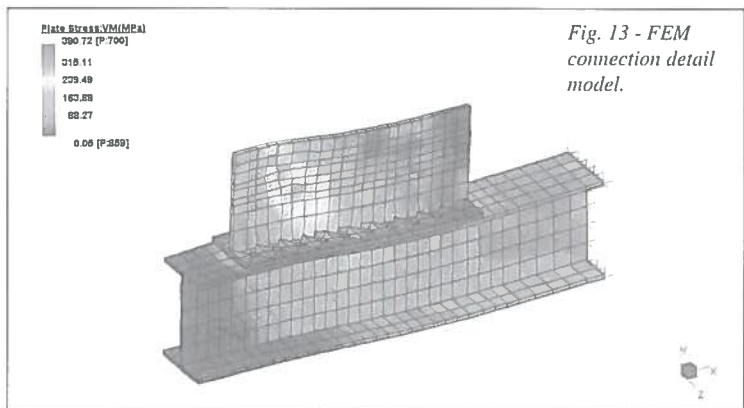


Fig. 13 - FEM connection detail model.

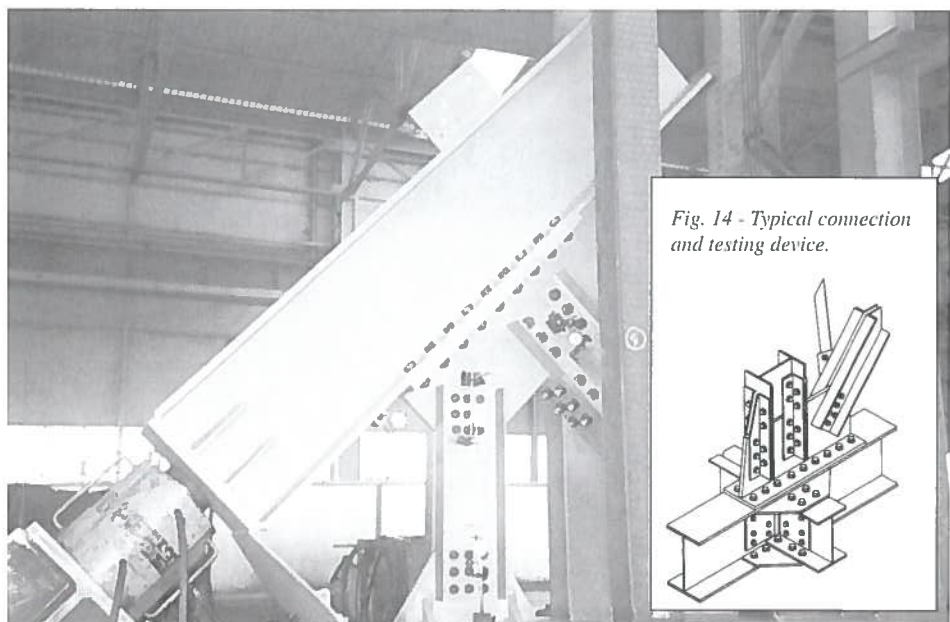


Fig. 14 - Typical connection and testing device.

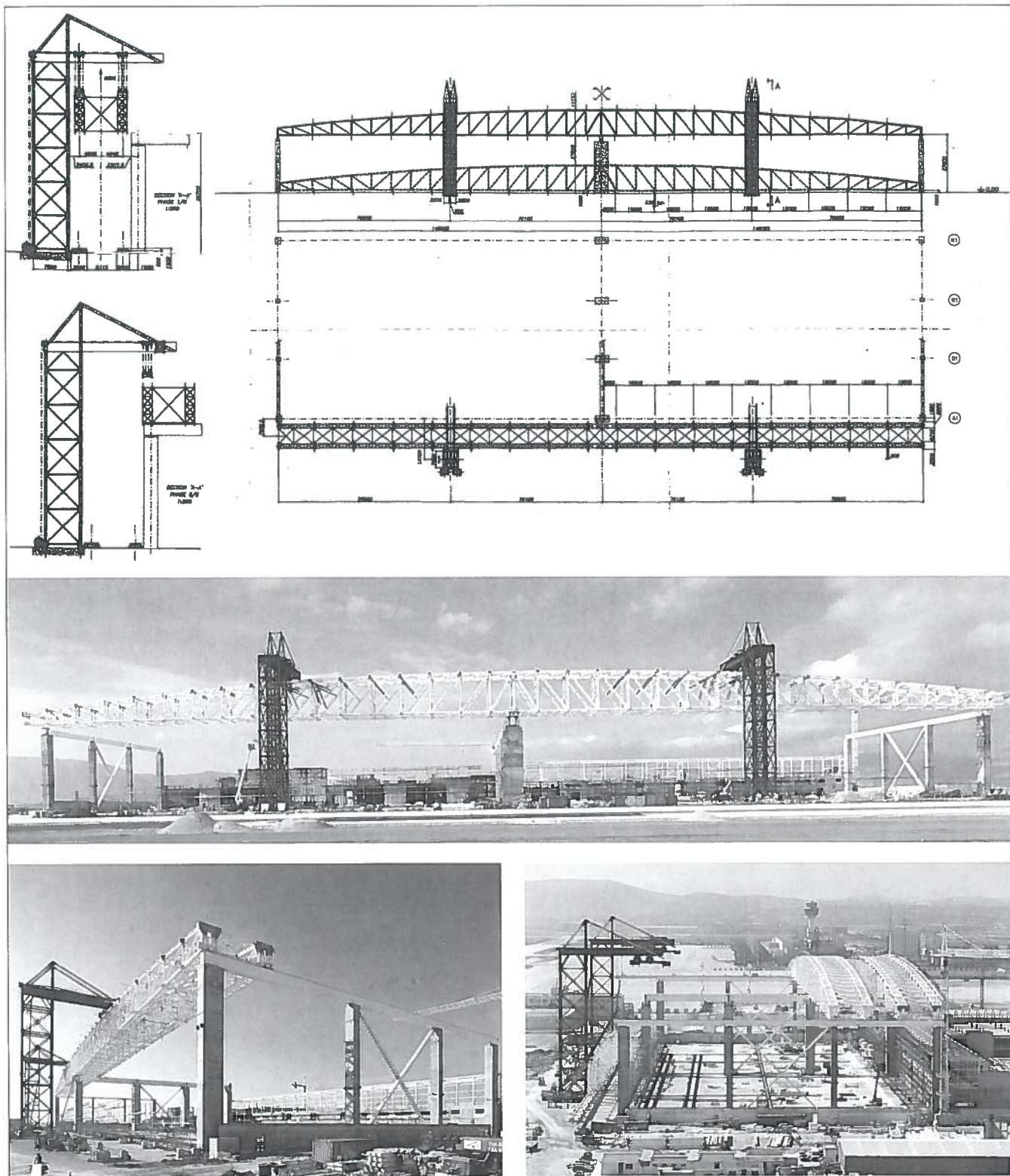


Fig. 15 - Scheme and erection of the first composite spatial truss frame.

- 2) of equivalent slenderness λ_{eq} ;
- 3) of compound slenderness λ_{comp} (assessed assuming $i_{comp} = (J_{y_comp}/(2A))^{1/2}$ where $J_{y_comp} = 2(J_y + A(L/2)^2)$) or the moment of inertia at the level considered of the section made up of two currents at a distance L_i);
- 4) the ratio σ_{crit}/f_y associated with λ_{eq} ;
- 5) the ratio σ_{crit}/f_y associated with λ_{loc} ;

Detail design assisted by testing

During the workshop detail elaboration a modification of the connection, to speed up construction by

automatic cutting and drilling machinery, was proposed by the steel supplier and validated by the designers after analytical verifications and testing assistance as recommended by EC1 and EC3 point 8. Concerning the theoretical simulation a FEM analysis was produced by the contractor in order to obtain the level of peak stresses (fig.13) The results of the analytical simulation were controlled by testing a typical connection (fig.14)

ERECTION PROCEDURE FOR BUILDINGS 4 AND 5

The assembly and erection of the two main bays of the hangar was accomplished with the following procedure:

- Phase 1. General excavation and construction of the drainage system.
Construction of the primary steel elements and transportation to the site.
- Phase 2. Construction of the piles, pile caps and underground substructure.
- Phase 3. Construction of the concrete columns.
- Phase 4. Assembly and erection of the main steel box girders.
- Phase 5. Assembly of the temporary lifting towers.
- Phase 6. Assembly on the ground of the main twin trusses two by two.
- Phase 7. Lifting the two main twin trusses, small horizontal movement and lowering on the main box girders.
- Phase 8. Sliding of the two main twin trusses on

rails on the main girders to their final position and fastening.

- Phase 9. Erection of the bracings (vertical and horizontal).
- Phase 10. Erection of the side walls.
- Phase 11. Erection of the catwalks, the runway beams for the cranes.
- Phase 12. Erection of the doors.
- Phase 13. Erection of the final covering.
- Phase 14. Completion of the slab on ground.

In the main beams there was an initial negative deflection equal to the deflection of the permanent loading (approximately 28 cm).

Most of the electromechanical equipment was positioned on the main trusses during the assembly phase, while still on the ground.

ACKNOWLEDGEMENTS

The authors thanks Engineers R. Trevisan and S. Pinardi for the outstanding contribution.

REFERENCES / DATI RELATIVI ALL'OPERA

Main contractor / Appalto generale: J/V Helliniki Technodomiki S.A. - AEGEK - AKTOR S.A.

Architectural design / Progetto architettonico: H. Bougadelli and Associates AETER

Structural design / Progetto strutturale: Ph. Karydakis - J. Mavrakis and Associates Structural Design Office Ltd. and Studio Tecnico Majowiecki.

Electromechanical design / Progetto elettromeccanico: by AKTER Ltd.

Steel constructor / Costruttore carpenteria metallica: Cimolai spa, Pordenone

Design of erection system / Calcolo del sistema di montaggio: Studio Tecnico Romaro, Padova

Ing. Phaedon Karydakis,
civil engineer NTUA, M.Sc. D.I.C., Ph. Karydakis – J. Mavrakis and associates ltd

Massimo Majowiecki, Prof. dr. ing.,
DISTART, University of Bologna