Alinghi Base for the 32nd America’s Cup, Valencia, Spain

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Introduction

On 2 March 2003, Team Alinghi went down in history for defeating Team New Zealand to bring the America’s Cup (AC) to Europe for the first time in over 150 years. After that victory, the Société Nautique de Genève, Switzerland, on whose behalf Team Alinghi competed, together with the Challenger of Record, the Golden Gate Yacht Club, announced the creation of America’s Cup Management (ACM), an independent company mandated to organize the 32nd AC. On 26 November 2003, ACM announced that Valencia, Spain, would host the 32nd AC. The programme for this edition of the AC included an ambitious 4-year schedule of regattas starting in 2004 and culminating in the AC Match, to be held between 23 June and 7 July 2007.

As a result and within the framework of a more ambitious urban development project, the Inner Harbour of the Port of Valencia has been reconditioned for the AC event. The infrastructure works included, among others, 12 bases that would serve as homes before and during the regattas for the defender, Team Alinghi, and each of the 11 challengers from all over the world. In an environment in which the most recent scientific and technological innovations are put immediately into practice in pursuit of lighter and faster boats, the challenge was to deliver sound buildings based on modern structural solutions, in which the design objectives of safety, serviceability, economy and elegance were to be achieved mainly by means of coherent conceptual design, efficient use of materials and good detailing. This paper describes the structural concept underlying the Alinghi Base, along with the solutions for a number of structural details and certain constructional considerations.

Keywords: Alinghi Base; composite structure; hollow section; bolted connection; composite joint; glass facade.

Architectural Design

Inasmuch as the various AC participants’ needs and preferences for their bases were unknown during the design stage, the organization decided to provide the shell only, leaving the outfitting to each individual team. Further to Team Alinghi’s service criteria, its base is double the size of the standard base defined by the AC organizers. From the outside, the viewer sees a prism-shaped building, 68 m long, 39 m wide and 13,5 m high (*Fig. 1*). The sea-side south facade has two large entrances to the boatshed, positioned between axes 2’ and 4’ and 5’ and 8. A third opening (between axes 12 and 13) corresponds to a corridor that crosses the entire building to provide access to the jetty from the building’s street-side facade. The rest of the south facade is panelled or glazed, the latter particularly on the second storey, where the 4,5 m wide balcony off the VIP lounge is accessed through glass doors. This balcony also stretches around half of the length of the east facade, likewise windowed at this level. The street-side facade consists in a slanted glass wall. The total usable area, nearly 7000 m^2 counting the four storeys (including the accessible roof) into which the building is divided, is occupied by offices, meeting rooms, a gymnasium, a restaurant and a shop, in addition to the aforementioned boatshed and VIP lounge. The roof is also fitted with a bar.

Conceptual Design

Overall System

The framework envisaged for the Alinghi Base was fairly conventional, with steel columns, composite beams and composite slabs (*Fig. 1*). The design called for continuous columns, from the foundations to the roof (*Fig. 2*). Longitudinally, they were spaced from 5 to 18 m, whereas transversally the standard spacing was 5 m, except in the corridor between axes B and C (*Fig. 1(c)*), where columns were spaced at 5,5 m for architectural reasons. Finally, the corridor running from axis A’ to axis B was to be bounded by the slanted glass facade, varying in width from 0,7 m on the ground to 2,9 m on the second storey. On all storeys, the main beams were to run parallel to the longitudinal facades of the building. The distance between these beams was to be spanned by a composite slab with shaped steel sheeting to avoid the need for any transverse beams, except around the edges to tie the structure together during construction. This layout, free of any secondary steel girders,

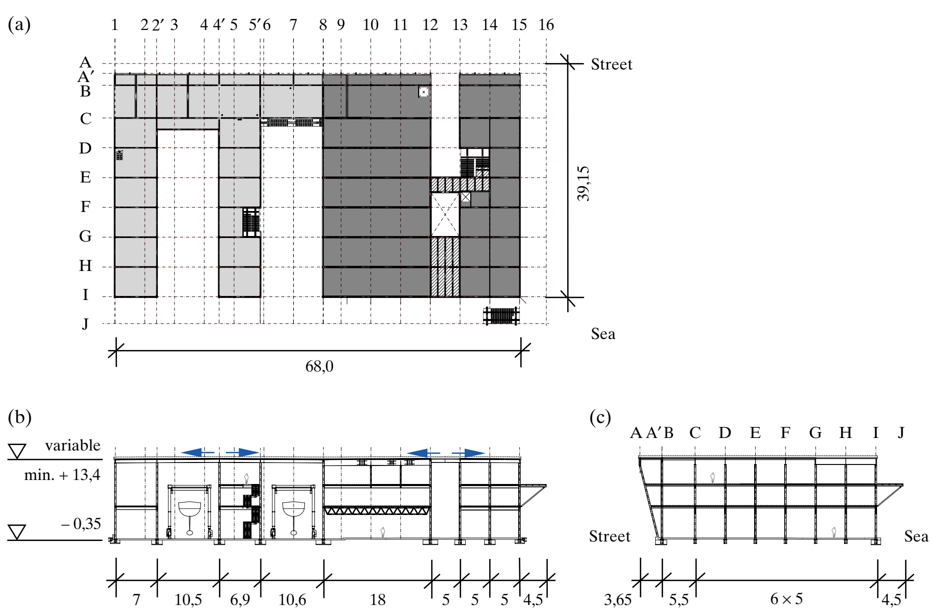

Fig. 1: Alinghi Base structure (a) plan view at first storey level; (b) longitudinal section; (c) cross-section (Units: m)

Fig. 2: Alinghi Base during construction, view from the jetty

allows to minimize the number of elements and on-site connections.

A 640 m^2 column-free area was envisaged for sail drying on the ground and first storeys, between axes 8 and 12 and B and I, respectively (*Fig. 1*). Owing to spatial limitations in the second storey that affected the composite system, the longitudinal beams had intermediate supports in the form of hangers connected to the roof beams, the strength and stiffness of which were to be enhanced accordingly. By contrast, the height available in the first storey system sufficed to solve the 18 m span length between the columns at axes 8 and 12 with simply supported composite truss girders spaced at 5 m centres.

Although Valencia is not a seismic region, the building was to be fitted with a bracing system to transmit the horizontal forces due to wind action and sway imperfections to the foundations, a solution that entails a series of advantages over the sway frame design. Both horizontal displacements and column buckling lengths can be reduced, the former being particularly important to ensure the appropriate behaviour of the brittle glass facade. Of particular relevance in this case was the fact that the bracing system solution adopted implied a simplification of both the beam-to-column composite joints, thereby freeing the full bearing capacity to accommodate vertical loads, and the foundations, to which no bending moments were to be transmitted. The bracing systems were to be built into facades in the form of diagonals in the windowless areas to minimize visual impact, using the adjacent columns as chords. On the sides of the building, the bracing systems were therefore positioned between axes C and D, while on the slanted facade they were set between axes 1 and 2, for here only the area bounded by axes 4' and 15 was to be glazed. The composite slabs would act as diaphragms, transmitting any horizontal forces to the bracing systems.

Structural Members

The engineer's design called for columns made of square, 8 mm steel hollow sections measuring 300 mm on each side, which up to the first storey would be filled with concrete. The latter was a safety measure adopted to increase the fire resistance of the columns, in light of the type of work conducted in the boatshed. Wherever possible, standard rolled steel I sections were used for the composite beams. With a standard spacing between beams of 5 m, IPE 400 shapes were used in the first storey areas subject to small live loads (characteristic value = 2 kN/m^2) and having a maximum span length of 7 m (between axes 1 and 2', *Fig. 1(a)*). In all areas with live loads ranging from 3 to 5 kN/m^2 and span lengths of up to 10,6 m (between axes 5' and 8 at roof level, *Fig. 1(b)*), HEA 240, 360 or 450 shapes were used, depending on the combination of live load intensity and span length.

Thanks to the use of intermediate hanger supports, the second storey composite beams that were to span the 18 m between the columns at axes 8 and 12 could be designed to take HEA 360 sections (*Fig. 1(b)*). For the composite roof beams with span lengths of 18 m, however, the design specified 1005 mm high steel I girders to withstand the loads generated by the accessible roof and suspended second storey. These beams would also have to accommodate large web perforations for mechanical and electrical services, with heights of up to 50% of the girder height. As the first storey beams between axes 8 and 12 were to be exposed, composite truss girders were to be used in this area for reasons of aesthetics. With a total height of 1270 mm, these trusses would consist in HEB 180 shapes on the top and bottom chords and HEB 120 sections for the diagonals.

In each composite beam, the composite slab would constitute the top flange, to which the respective steel girder was to be connected. The total slab depth envisaged was either 0,12 m (first storey between axes 1 and 8) or 0,18 m (all other areas), depending primarily on the magnitude of the live loads. The viability of the composite slabs, with span lengths of up to 5,5 m and the aforementioned overall depths – for slenderness ratios of 46 and 30, respectively – would be dependent upon the use of lightweight aggregate concrete, LC, with a density of 1800 kg/m^3 and a characteristic compressive strength of 40 N/mm^2. Given the seaside location and the presence of large openings in the south facade of the building, the admissible chloride content established was 0,1% of the bulk weight of the binder. While there are mandatory standards in prestressed concrete, such a requirement is not normally applied in composite structures. The overall depth of the steel sheeting to be used in the composite slabs positioned transversally to the beams was 75 mm. With the span lengths adopted, the steel sheeting available would have been unable to transmit the horizontal shear at the interface between the sheet and the concrete by means of mechanical and frictional interlocking alone. Consequently, the composite beam stud connectors were to be welded through the sheeting as end anchorage to ensure the composite behaviour of the steel sheeting and the concrete.

Joints

Along with shear, negative bending moments due to predominantly static loads constitute the main action effects at the beam-to-column connections; this led to the adoption in the design of a semi-rigid, partial-strength composite joint (*Fig. 3*) that attained both design goals, structural robustness and the efficient use of materials. Shear was to be transmitted across a bolted double lap joint connecting the web of the steel shapes to a gusset welded in an insert cut into the hollow section steel column. Connections between steel girders and columns were to be bolted to shorten structural assembly times. During the final stage, the reinforcement in the composite slab, in turn, would transmit the tensile forces generated by the negative bending moments. The compression forces arising from those moments would be transmitted by contact pressure across a contact plate between the bottom flange of the steel profile and a horizontal stiffener welded to the outside of the hollow steel column.

The cladding on the glass facade was to be attached directly to the slanted columns rising from the foundation to the roof (*Fig. 1(c)*). The objective sought in the design of these columns was maximum slenderness. Consequently, the only loads to be transmitted to these members, other than wind pressure, were the vertical loads from the roof. The corridors running from axes A' to B on the first and second storey slabs were conceived as cantilevers. At the same time, these slabs were to act as lateral restraints for the columns. Therefore, the connections between the slanted columns and the slabs were designed as pins in long slotted holes, with the axes of the slots perpendicular to the direction of the transfer of wind loads (*Fig. 4*). With this scheme, the span length for transmitting wind loads would be equal to the storey height. The foregoing would also be applicable to both the in- and out-of-plane buckling lengths for the slanted columns. With such arrangements, the slanted columns could be built with rectangular hollow sections measuring 150 mm × 250 mm, with the strong axis oriented to resist the wind pressure; the design slenderness ratio was 80.

Construction

In light of the tight construction schedule, the steel structure subcontractor suggested a fundamental change in the original detailing. To shorten construction times, the design beam-to-column joint configuration (*Fig. 3*) was replaced by a welded solution. Welded connections between steel I girders and hollow section steel columns, however, may lead to failure of the thin column face due to the appearance of plastic hinge lines, both during the intermediate (joint between steel members) and final (composite joint) stages of the works. Moreover, when this modification was introduced, it was too late to change the column cross-sections (e.g. open instead of hollow). Measures had therefore to be adopted to prevent column-face-failure during the intermediate and final stages of construction:

– The hollow section steel columns were converted to composite columns in which the concrete infill would stiffen the slender steel constituent plates. This would prevent plastic failure due to concentrated compression forces induced by hogging bending moments and transmitted across the steel shapes. This measure would also enhance column fire resistance.

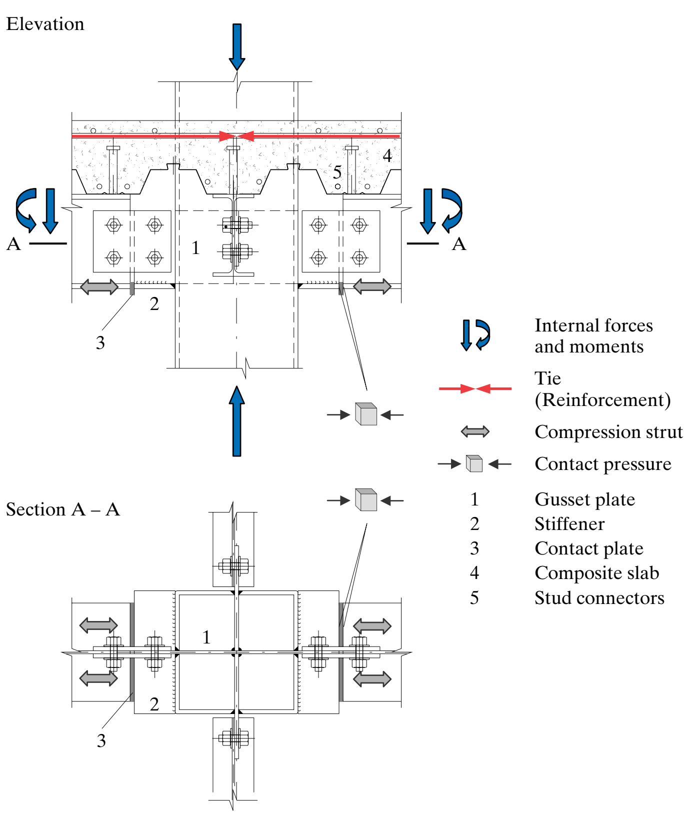

Fig. 3: Four-sided, semi-rigid composite joint for beam-to-column connections

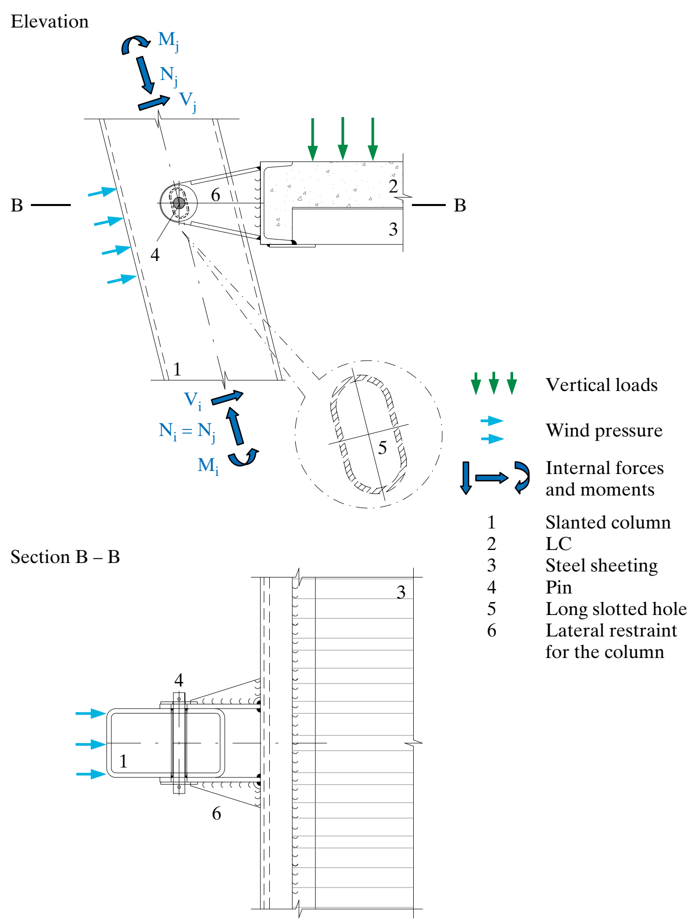

Fig. 4: Slanted facade column-to-slab connection via pins in long slotted holes, in which the slab restrains the column laterally

– In the final stage, the reinforcement in the composite slab would transmit the tensile forces generated by the negative bending moments. Owing to welded connection stiffness, however, bending moments would also appear during construction, before the reinforcement would be operational. The moment-induced tensile forces might therefore lead to plastic deformation on the column face, inasmuch as the concrete infill would act as a stiffener for the compression forces only. An external ring stiffener positioned on the same level as the top flange of the steel shape had therefore to be devised to prevent the appearance of plastic hinge lines (*Fig. 5*).

This example shows how last-minute changes to the original design, introduced during construction to accommodate contractor preference, may have consequences that tend to cancel out the presumed advantages of the change. Indeed, self-compacting concrete (SCC), was required to convert the hollow section steel columns into composite members. The determination of the appropriate concrete mix and procedures for placing the concrete called for preliminary testing that took up a certain amount of the already limited time available for construction. Similarly, installation of the aforementioned ring stiffeners slowed erection of the steel structure. Nonetheless, except the change in the original beam-to-column connection detailing and the follow-up changes described above, the structure was built to design and completed within the demanding construction deadlines.

Conclusion

The above description of the run-up to the construction of the Alinghi Base exemplifies the importance of conceptual design, for a structure's serviceability, reliability and economics depend on the solutions adopted at that stage. When the conceptual approach is well thought out, the design will undergo only minor changes in subsequent stages. The success of a structural design depends on close three-way cooperation between owner, architect and engineer throughout, starting with the definition of the service criteria.

Close co-operation among all stakeholders is also of cardinal importance during construction, to ensure that structures are built to design. Changes made during the construction stage are usually more a reflection of contractor preference or other circumstantial factors than of actual technical or practical needs, and may have detrimental side effects. Given the interaction between the design, construction, technical and aesthetic quality – not to mention the economics – of a structure, designers must take an active role in construction.

In the case of the Alinghi Base a sound solution was designed, thanks to the adoption of a coherent and modern structural approach that ensured the efficient use of materials. The decision to integrate all members into the overall structural system entailed an economic quantity of structural steel required, on the order of 40 kg/m^2 (referred to the total usable area). Compared to the structural steel deployed in the standard bases defined by the AC organizers, a reduction on the order of 45% has been achieved. The resulting savings may have been invested in the development of the Alinghi Team's new boat, *SUI 100*, thereby contributing to its successful defence of the America's Cup against the winner of the challenger races, Emirates Team New Zealand.

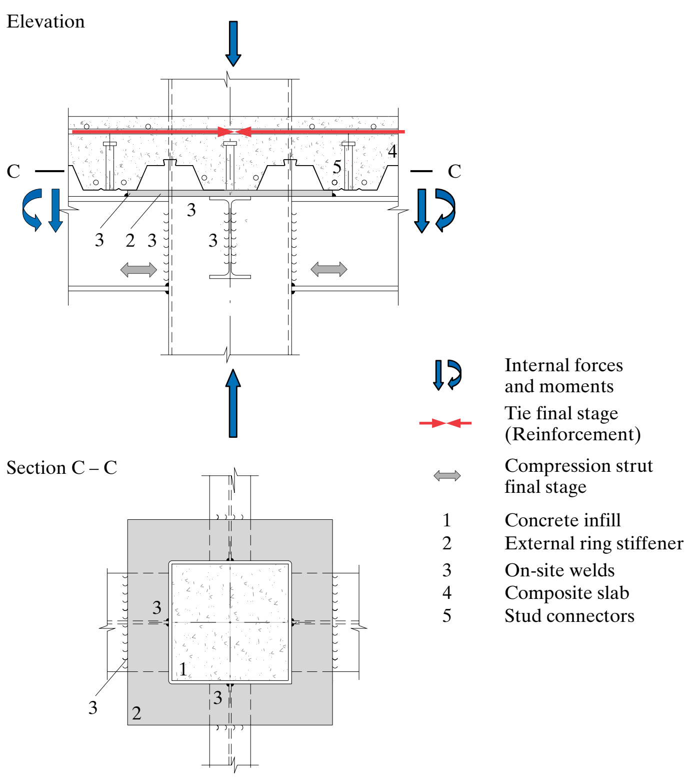

Fig. 5: Welded beam-to-composite column connection with external ring stiffener

SEI Data Block

Owner:
– Structure
City of Valencia
– Facades, M&E, Finishes
Team Alinghi

Architect:
Areas Ingeniería y Arquitectura, Valencia, Spain

Structural Engineers:
P. Tanner and J. L. Bellod, Cesma Ingenieros, Madrid, Spain

Main Contractors:
– Structure; Joint Venture Dragados–Sedesa–Drace, Spain
– Facades, M&E, Finishes
Nüssli, Switzerland

Steel Structure Subcontractor:
Lems, Valencia, Spain

Structural steel [t]	280
In situ concrete [m^3]	
– LC in composite slabs	783
– SCC in composite columns	76
Usable area [m^2]	7 000
Total cost of structure [EUR millions]	2,1
Service Date	October, 2005