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Semi-continuous beam-to-column joints at the Millennium Tower in Vienna, Austria

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SEMI-CONTINUOUS BEAM-TO-COLUMN JOINTS AT THE MILLENNIUM TOWER IN VIENNA, AUSTRIA

Gerald Huber [1]

Abstract

The Millennium Tower is situated to the north of the center of Vienna. With a height of 202 m it is the highest building in Austria. Realization was improved by new methods. The tower is a typical example of mixed building technology, combining composite frames with a concrete core. Special attention has been paid to the moment connections between the slim floors and the column tubes resulting in a drastically reduced construction time and thin slabs. The semi-continuity has been considered in the design at ultimate and serviceability limit states.

1. INTRODUCTION

1.1 The Project "Millennium City"

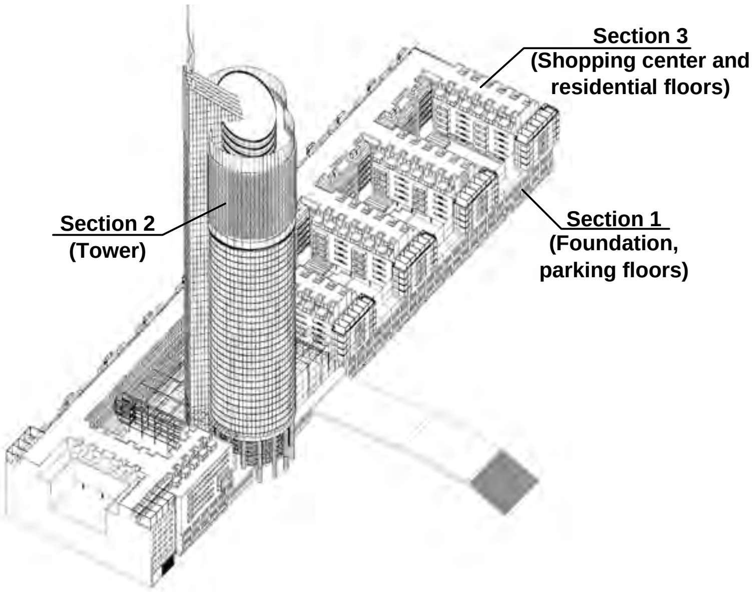


Fig. 1 Millennium City, Vienna

In 1996 the Vienna municipal council agreed to the "Millennium City" project of "Stumpf Immobilien- und Wohnungseigentum GmbH" (Vienna) with residential blocks (37.000 m²), a commercial area (25.000 m²) and an office tower (38.000 m²) planned by the team of architects Peichl-Podrecca-Weber (Vienna). The construction of this "City in the city" (*Fig. 1*) started in 1997 on a ground area of 15.500 m² - conveniently placed what regards transport facilities - with a capital expenditure of about 145 Million Euro (1).

The overall project is separated into three sections: Section 1 includes four basements with a parking area for 1.500 cars and the tower foundation plate on 151 bored piles with a length of 25 m. Section 2 is the tower itself with 50 upper floors and an antenna of 30 m. Section 3 contains two shopping floors and six residential floors and has been erected simultaneously to the tower.

1.2 Millennium Tower

With a total height of 202 m it is the highest building in Austria. Work started in May 1998. By realizing 2½ up to even 3 floors per week the building shell was already completed in January 1999 after only 8 months of construction. The final handing over to the owner happened in April 1999. The plan of the tower with 1.080 m² consists of two overlapping circles for offices and a concrete core which contains the elevators and stairways, the foyer, archives and additional office area in the so-called tower back (*Fig. 2*). The core has been realized in conventional concrete building technology to transfer vertical forces and all horizontal forces due to wind and earthquake. At the other hand the tower circles are formed by concentric composite frames, which are only designed for vertical forces. The combination of concrete and composite building technology finally results in an overall "mixed building".

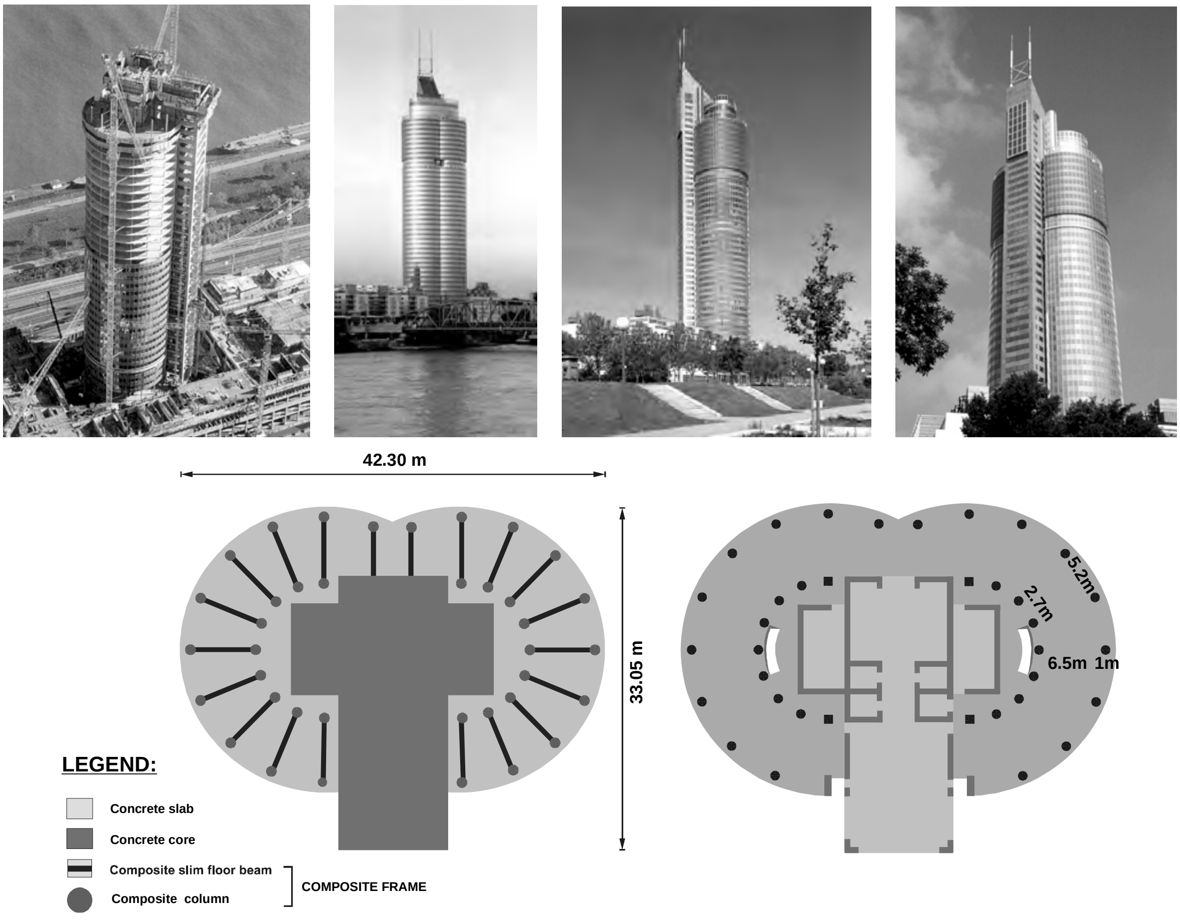


Fig. 2 Millennium Tower

The demands for an extremely fast and weather independent erection, very thin slabs (reduced dead load and lower facade costs) with a plane ceiling (easier installation) and very slender columns called for an ingenious solution, which included the following building innovations: Composite slim floor beams fully integrated into the thin slabs, moment-resisting (semi-continuous) joints enabling a frame action between the beams and columns and a new type of shot-fired shear connector within the composite columns. The capital expenditure for the tower shell amounted to 12,5 Million Euro based on 1.500 tons of constructional steel, 2.500 tons of reinforcing steel and 15.000 m³ of concrete.

2. COMPOSITE FRAMES

2.1 General

The vertical forces of the two overlapping tower circles are carried by 20 external and 18 internal columns in a concentric distance of 6,5 m (*Fig. 2*). The external columns are located 1 m inside of the facade with a transverse distance of 5,2 m. The space between the internal columns is 2,7 m. The interplay between an external column, the external joint, the slim floor beam, the internal joint and the internal column forms a frame system (*Fig. 3*) with the effect of a considerable reduction of sagging moments, deflections and vibration of the slab. The frame capacity of transferring horizontal forces additionally to the concrete core has not been taken into account. The big number of analogous joints obviously justifies a very detailed planning to optimize the advantage of moment connection with regard to the erection time and costs.

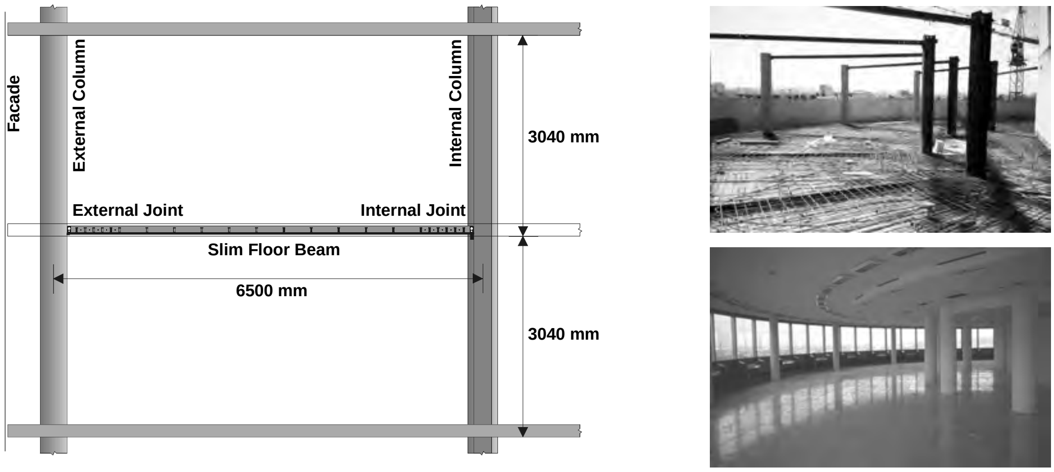


Fig. 3 Composite frames with semi-continuous joints

2.2 Beam cross section

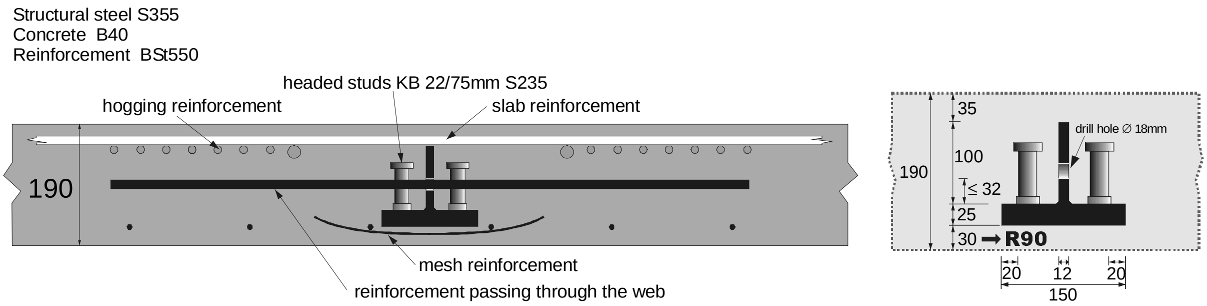


Fig. 4 Composite slim floor beam

The composite slim floor beams (*Fig. 4*) are built of welded T-shaped steel sections and a concrete slab with minimum sagging reinforcement and a considerable amount of reinforcement in the hogging region within the effective width. The shear connection is provided by headed studs. The non-linear characterization of the sagging cross section considering partial shear connection and that of the hogging cross section including the effect of tension stiffening has been performed with the software developed in (2) basing on Eurocode 4.

2.3 Column cross section

The demand for very slender columns led to composite sections with steel tubes and additional steel cores, both S355. The diameter of the tube, the size of the core and the concrete grade have been adjusted to the actual stresses depending on the floor number. As shown in *Fig. 5* the diameter of the external columns varied from 324 up to 406 mm. To ensure optimal filling of the remaining space between tube and core self-compacting concrete of grades B40 to B60 has been used. To defuse the severe problem of different creep and shrinkage between the composite columns and the concrete core due to a different steel-to-concrete ratio the internal columns – closer to the concrete core – have been realized as concrete-encased rolled I-sections with a higher concrete percentage. Their diameter is 450 to 500 mm (*Fig. 5*). The normal stresses in the columns have been determined with influence areas basing on plastic redistribution. Except for the top columns the normal force is clearly dominating in comparison to the bending moments resulting from the frame action due to the semi-continuous joints. Design calculations under normal conditions and in case of fire (R90) have been based on Eurocode 4.

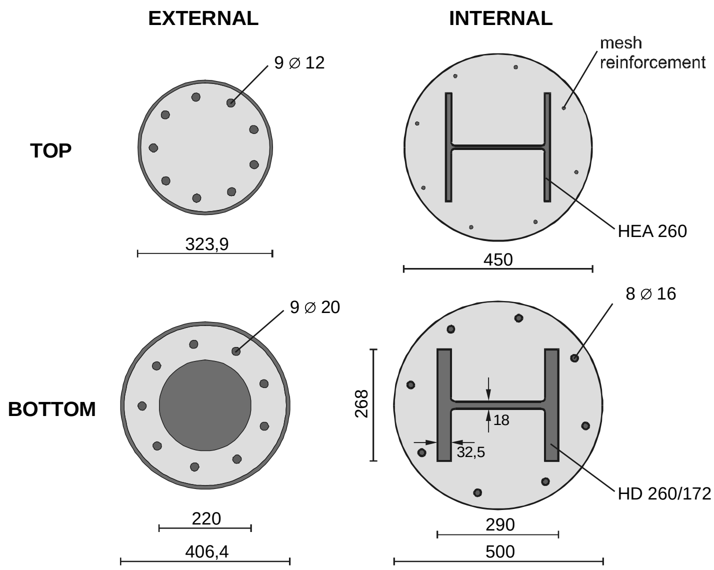


Fig. 5 Column cross sections

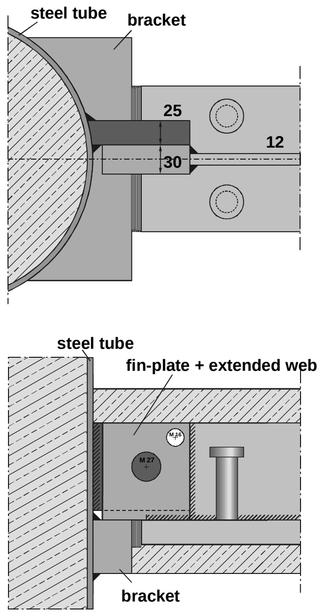


Fig. 6 Support of beam at external column

The vertical support forces of the beams are handed over to the column steel tube via a welded bracket (in cold stage) and a fin-plate (in case of fire) (*Fig. 6*). Parts of these concentrated forces then have to be passed to the chamber concrete and further to the steel core. Instead of conventional welded studs - as a novelty - shot-fired nails and bolts (*Fig. 7*) have been applied (3)(4).

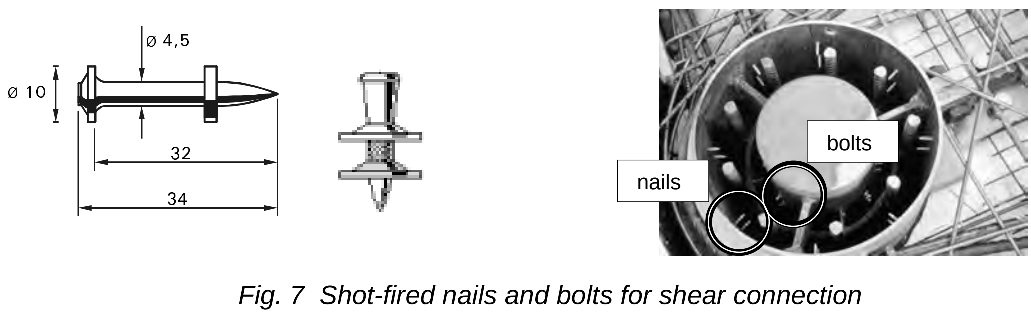


Fig. 7 Shot-fired nails and bolts for shear connection

3. SEMI-CONTINUOUS JOINTS

3.1 General

The use of composite slim floor beams in combination with composite columns solves two problems simultaneously which would appear in conventional concrete joints: punching and a low moment resistance combined with a brittle failure due to the limited load introduction of concrete in compression (*Fig. 8*).

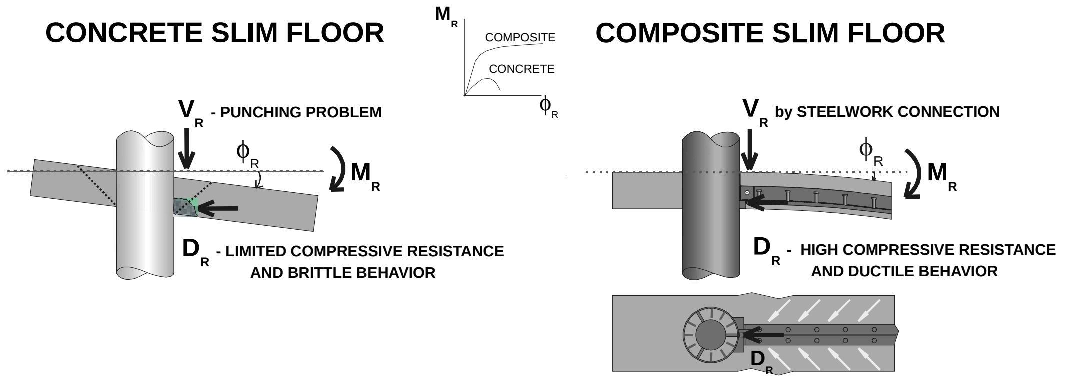


Fig. 8 Comparison between concrete and innovative composite slim floors

3.2 Joint configuration

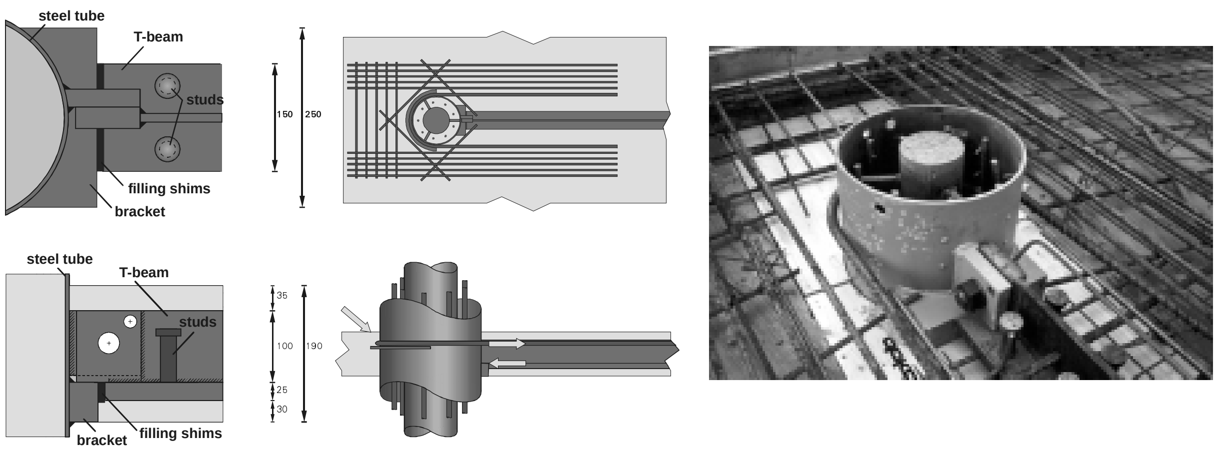


Fig. 9 Regular external joint configuration

Fig. 9 shows the actual configuration of a joint between an external column and a regular slim floor at the Millennium Tower. The compressive force is transferred from the beam flange via clearance filling shims into a bracket which is welded to the column tube. From there it spreads vertically and horizontally into the hollow steel section, the chamber concrete and the steel core. The tensile force of same size goes through the beam's shear connection into the hogging reinforcement. A reinforcement U-bar of 20 mm diameter goes around the column in direct contact. A saddle already welded in the shop ensures the exact location of this bar and therefore the joint's lever arm. The remaining restraint reinforcement (7 bars of 12 mm at both sides of the column) extends into the cantilevering part of the slab. Together with the transverse reinforcement and concrete struts a truss is built handing over the tension force into the column via bearing pressure. Especially in the case of such slim floor joints with a small lever arm, its constructive observation is crucial as a deviation of some centimeters already would cause a significant loss of stiffness and moment resistance.

Such a combination of a U-bar and a reinforcement truss can only be realized in the case of a cantilevering slab. As for the lower five floors the facade should be located directly behind the column an alternative to the regular joint had to be developed with additional U-bars resulting in a necessary slab outstand of only 6 cm (*Fig. 10*). To avoid splitting due to the arrangement of three U-bars an additional top saddle has been provided. By optimizing the reinforcement layout the overall response of these different external joint configurations is nearly similar in view of stiffness and resistance.

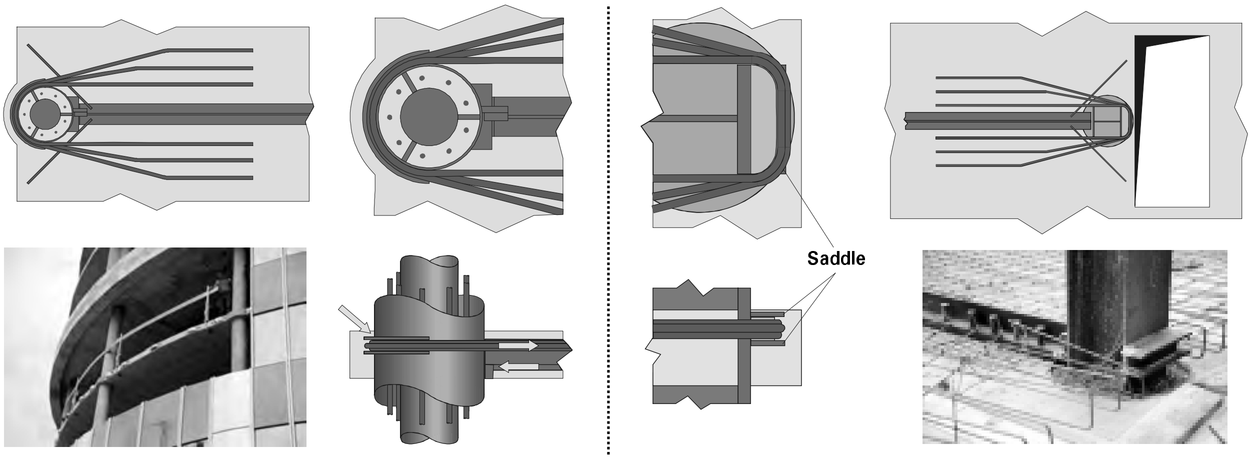


Fig. 10 External and internal joint configuration without cantilevering slab

3.3 Joint characterization

For the regular external joint configuration (*Fig. 9*) the following components can be identified contributing to the overall joint behavior. Component C1 is the redirection truss within the cantilevering slab set together by the longitudinal and transverse reinforcement in tension and diagonal concrete struts bearing to the column. The reinforcing U-bar, component C2, acts in the same way as C1 anchoring the tension forces into the column. So C1 and C2 are sharing the overall tension force as two parallel components. The bolt of the fin-plate (*Fig. 6*) is only used for vertical shear transfer in case of fire. An interaction in tension due to the moment-connection is prevented by hole clearance. Component 3 represents the slip between the T-shaped steel beam and the concrete slab due to incomplete shear interaction. The compression region is formed by the components C4, C5 and C6 reflecting the compression in the beam flange and the filling shims, the load introduction into the steel tube via the bracket and the stiffening effect of the chamber concrete within the tube. For such an edge joint the beam's hogging moment is not balanced by a similar connection on the other side and therefore the full restraint moment has to be transferred into the column. The concentrated load introduction in tension and in compression are then causing local shear (C7) and bending (C8) of the steel tube, reinforced by the chamber concrete (C9). *Fig. 11* gives an overview of the actual components and the key values of their individual behavior in view of initial stiffness (c or S) and design resistance (F_{Rd} or M_{Rd}) gained from analytical models according to the specified references. The detailed formulae can also be got from the example calculation in (5).

To get the overall moment-rotation response of the connection the influences of the individual components C1 to C6 have been assembled fulfilling equilibrium and compatibility according to the component model shown in *Fig. 12*. Simultaneously with a considerable degree of moment connection the Millennium Tower joints proved to be easy to handle both in view of erection and characterization. As there is only one row in tension the component curves could easily be added step by step parallel and serial without iterations (*Fig. 13*) using the computer program CoBeJo (2) which would even enable an iterative assembly for up to 7 rows in tension.

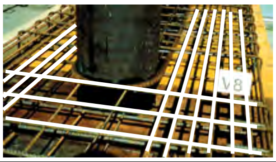
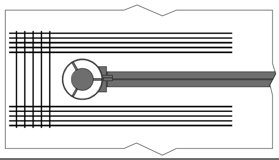
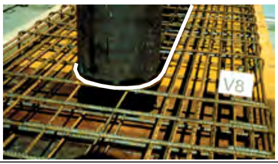
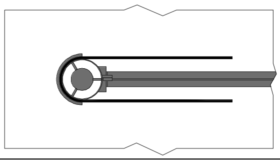
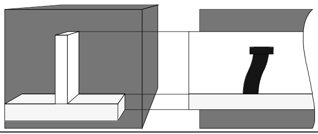
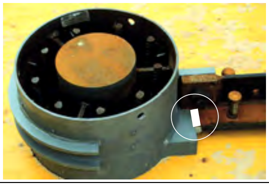
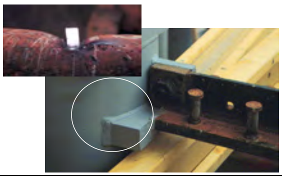
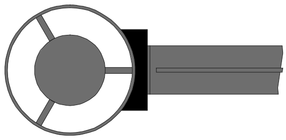
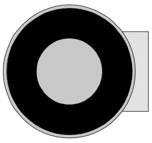
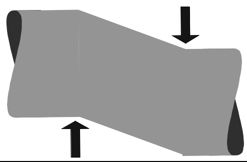
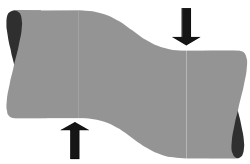
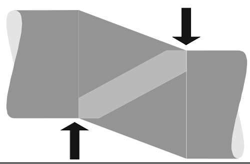
Components			Characterization
Connection (L)			
C1	redirection truss (longitudinal and transverse reinforcement plus concrete struts)	 	(6),(2),(7) c_1 = 430 kN/mm $F_{Rd,1}$ = 757 kN
C2	U-bar reinforcement	 	(6),(7) c_2 = 352 kN/mm $F_{Rd,2}$ = 300 kN
C3	slip due to incomplete shear interaction in the beam		(8) $c_3 = \infty$ kN/mm $F_{Rd,3} = \infty$ kN
C4	compression in the beam flange and shims		(6) $c_4 = \infty$ kN/mm $F_{Rd,4}$ = 1.331 kN
C5	load introduction into the steel tube via the bracket	 	(6),(9),(10) c_5 = 53 kN/mm $F_{Rd,5}$ = 147 kN
C6	stiffening in compression by the chamber concrete	 	(6),(9),(10) c_6 = 2.917 kN/mm $F_{Rd,6}$ = 1.100 kN
Shear panel (S)			
C7	shear deformation of the steel tube	 	(6),(9),(10) S_7 = 24,3 MNm $M_{Rd,7}$ = 56,2 kNm
C8	bending deformation of the steel tube	 	(6),(9),(10) S_8 = 1.272 MNm $M_{Rd,8}$ = 310 kNm
C9	stiffening in shear by the chamber concrete	 	(6),(9),(10) S_9 = 6,4 MNm $M_{Rd,9}$ = 41,6 kNm

Fig. 11 Joint components and their characterization

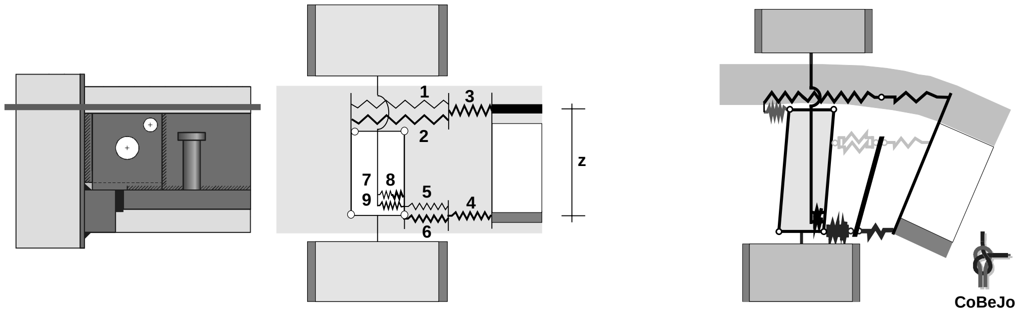


Fig. 12 Component model for assembly

Due to the very simple component interplay the key values of the connection's M-ϕ curve can even be estimated with the following formulae knowing that the lever arm z is 109 mm:

$$S_L = \left(\frac{1}{c_1 + c_2} + \frac{1}{c_3} + \frac{1}{c_4} + \frac{1}{c_5 + c_6} \right)^{-1} \cdot z^2 = 7.4 \text{ MNm} \qquad {}_{L,Rd} = z \cdot \min\left[(F_{Rd,1} + F_{Rd,2}); F_{Rd,3}; F_{Rd,4}; (F_{Rd,5} + F_{Rd,6}) \right] = 115 \text{ kNm}$$

The overall M-ϕ curve of the shear panel has been set together by the individual influences C7 to C9 in an analogous way; at the one hand for the overall curve as shown in *Fig. 14* and at the other hand estimating only the key values according to the following formulae:

$$S_S = \left(\frac{1}{1/S_7 + 1/S_8} + S_9 \right) = 30.2 \text{ MNm} \qquad {}_{S,Rd} = M_{Rd,9} + \min\left[M_{Rd,7}; M_{Rd,8} \right] = 98 \text{ kNm}$$

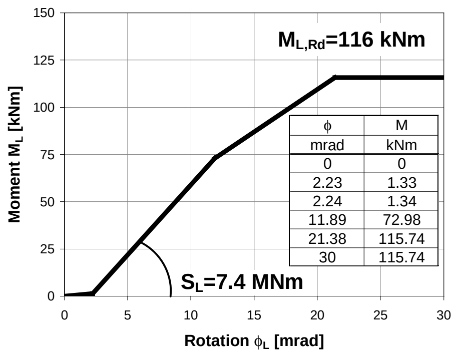


Fig. 13 M-ϕ curve of the connection (components C1 to C6)

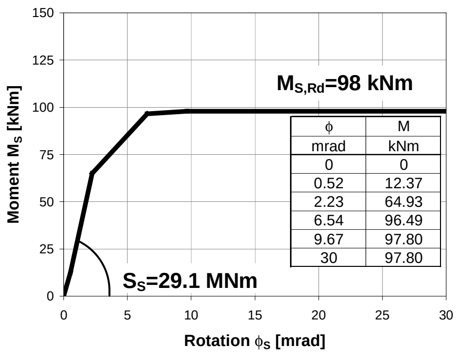


Fig. 14 M-ϕ curve of the shear panel (components C7 to C9)

3.4 Joint modeling

For a conventional joint configuration with double-sided connections the separate influences of the connections and the shear panel can be considered separately also in the global analysis. Neglecting the difference between the moment within the connection (M_L) and that of the shear panel (M_S) for the actual edge joint these two influences alternatively may be added in series resulting in a combined joint curve (*Fig. 15*) with the following key values:

$$S_j = \left(\frac{1}{S_L} + \frac{1}{S_S} \right)^{-1} = 5.9 \text{ MNm} \qquad {}_{j,Rd} = \min\left[M_{L,Rd}; M_{S,Rd} \right] = 98 \text{ kNm}$$

Fig. 16 shows the corresponding joint model with an infinitely rigid joint area and a rotational joint spring at the beam-to-column intersection point representing the overall joint deformability (11). The configuration of all joints at the Millennium Tower (external and internal, at the top and at the bottom) has been optimized in such a way that their response is nearly identical and therefore one single idealized bi-linear curve (*Fig. 15*) could be used for all joints of this building. A full-scale joint test impressively proved the analytical results.

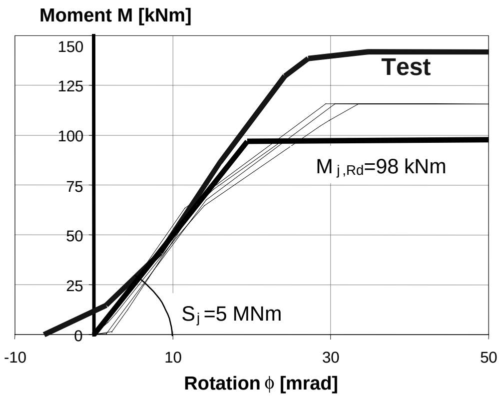


Fig. 15 Overall design joint curve

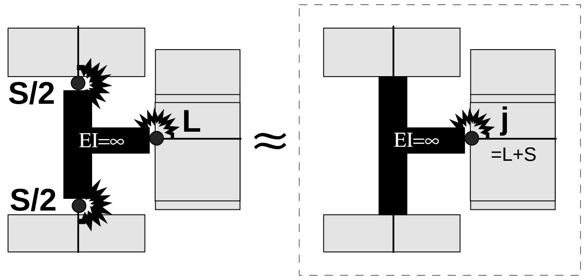


Fig. 16 Joint modeling

3.5 Global frame analysis

Knowing the response of the beams in sagging and in hogging, that of the columns and the joints out of the respective characterization the global frame analysis could be performed for ULS and SLS for all dead and imposed loads with the structural system shown in *Fig. 17* (12). A comparative calculation with perfect hinges or fully rigid restrains shows that the actual semi-rigid joints lead to deflections and bending moments quite in the middle between these borderline cases as an optimum between design calculation and economic detailing.

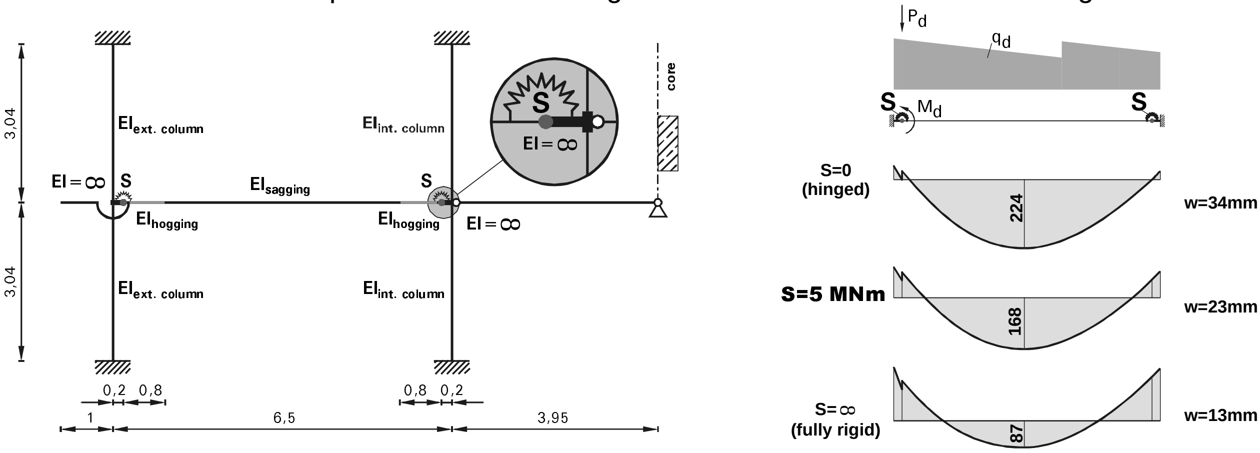


Fig. 17 Global frame analysis considering the joint behavior

CONCLUSIONS

It was shown that a simple support during erection can easily be transferred into a moment-resisting joint with considerable stiffness and resistance at final stage. Activating this frame action between beams and columns enables the realization of very slim floors under observance of ultimate and especially serviceability limit states. The analytical joint characterization was described in detail applying the component method. A full-scale joint test as well as measurements on site proved the calculated joint behavior. In addition the use of shot-fired nails and bolts as shear connectors within the hollow column sections helped speeding up the erection.

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