Rembrandt Tower, Amsterdam (The Netherlands)

Joris Fellinger, Leen Twilt, Pascal Steenbakkers
Centre for Fire Research TNO

1 GENERAL INFORMATION

Client:
Philips BV

Architect:
ZZ+P Architects

Planning of structural framework:
Samenwerkende adviesbureaus Amstelhoek

Executive company:
Sedijko

Fire protection expertise: (FSE approach)
Centre for Fire Research TNO

Processing time:
1996

Kind of building:
High rise office building

Total height:
135 m

Ground-plan:
32.4 × 32.4 m

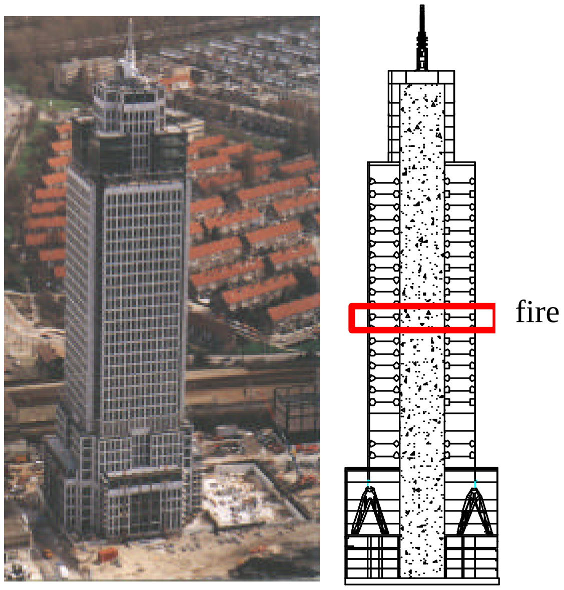

Figure 1:. Photo of the Rembrandt Tower in Amsterdam and the vertical cross section with the fire exposed structure.

2 INTRODUCTION

This high rise office building is situated in Amsterdam. The actual structural fire design is according to the existing Dutch building regulations, i.e. based on the traditional classification system. By way of alternative, the design approach, based on Fire Safety Engineering has been carried out. Some features & results of this approach are presented hereafter.

3 THE STRUCTURE

The building has a height of 135 m. It consists of a steel frame structure with steel columns in the façade, braced by a square concrete core in which the vertical transport systems are incorporated. See Fig. 1. The floors are made of composite decks using steel sheets supported on steel beams. The beams are simply supported, both at the concrete core and at the (continuous) columns. A typical storey is 3.4 m high and each storey consists of one fire compartment. See also Fig. 2.

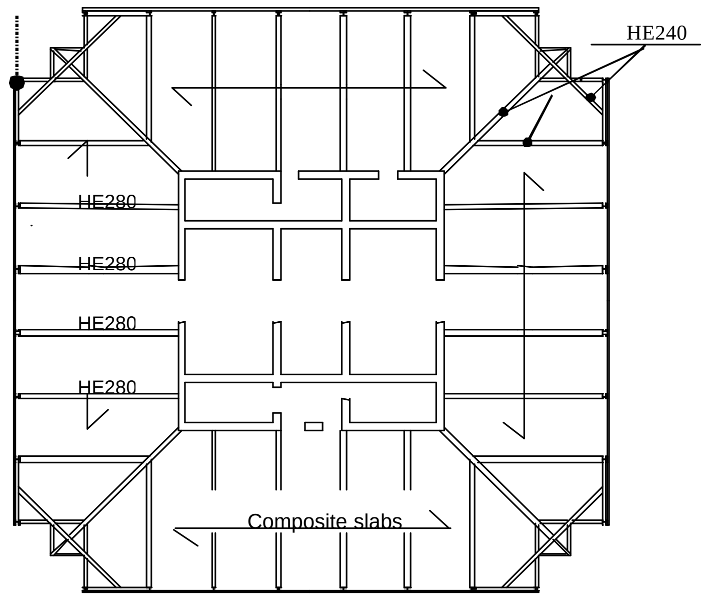

Figure 2. Typical floor plan.

4 CONVENTIONAL FIRE SAFTEY APPRAOCH

Dutch regulations require an equivalent fire safety level for building beyond 70 m as for buildings lower than 70 m. The functional requirement is that no collapse shall occur during the complete period of fire exposure. However, no method is prescribed to assess the safety level. In the design process, among other measures including a fully automated sprinkler system, a standard fire resistance of 120 minutes for the main load bearing structure was agreed between the designer and the local authorities. However, the standard fire resistance is determined on single structural members exposed to the standard fire. With such an approach uncertainties regarding the interaction between the structural member and the supporting construction are neglected. Loads and deformations introduced by restrained thermal expansion and the redistributions of applied loads can not be considered, while it is well known that these factors often overshadow other effects in fire exposed structures. Moreover, the beneficial effect of other mitigating measures on the structural fire safety can not be taken into account.

5 ALTERNATIVE FSE APPROACH

5.1 *Basis*

The objective of the FSE study is to get an understanding of the actual fire behaviour including the interaction between structural members. Moreover, the required thermal protection on the steel members was varied in order to get a cost effective design. For that purpose a FE model of the structural system of the tower was developed with the computer code DIANA, in which a fully developed fire was assumed in one fire compartment. The standard fire was replaced by a simulation of the fire development of a typical fire compartment in the building with the computer programme Ozone. The size of the braced steel columns reduces towards the top because of the lower loads. A simple fire analyses of the columns at each storey, based on the standard fire exposure, showed that the columns at the 21st storey were most critical. Therefore, this storey was modelled. See Figure 1.

5.2 *Fire development*

The fire development was modelled with Ozone. Since, most office spaces in the tower are furnished without partition walls, one big compartment was modelled of 32.4 x 32.4 m excluding the central core of 14.4 x 14.4 m. The actual thermal properties of the concrete core, the composite floors and the sandwich construction of the façade (steel sheet – mineral wool - granite) were modelled with nominal values for concrete, steel and mineral wool as given by the programme. The effect of the sprinklers was taken into account in a way as described in WP1. A big uncertainty is the ventilation resulting from the breaking of the windows. A small parameter study showed that the effect of the assumptions for the breaking of the windows on the temperature of the steel members in the compartment is relatively small. The results based on the assumption that all window break directly at the start of the fire were finally used as input for the FE model.

5.3 *Thermal response*

Separate FE models were applied for the determination of the time dependent and non-uniform temperature distribution of cross sections of the steel concrete composite slab, the regular HE280AA beams and the heavy corner beams HE240M. The temperature of the columns was obtained by Ozone, as a uniform temperature distribution could be assumed for these columns which were exposed from all four sides. See Fig. 3 for an example.

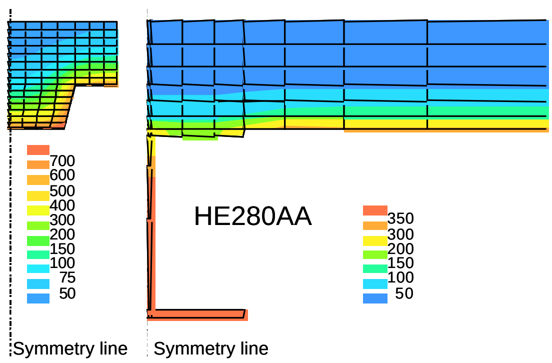

Figure 3: Temperature distribution in the composite deck after 75 min. fire exposure (left) and in the bare regular beams after 50 min.

5.4 *Mechanical response*

The entire floor of the 21st floor was modelled, including the columns. At the bottom side and top side, the columns were modelled with clamped supports, the top side allowing for vertical displacements. At the top side of the columns, the momentary part of the vertical loads of the rest of the building were applied, considering partial safety factors equal to unity. The beams, columns and reinforced ribs of the composite slabs were modelled with numerically integrated beam elements based on the Mindlin-Reissner theory. The reinforced concrete deck was modelled with curved shell elements. The steel sheet was modelled as reinforcement, considering a separate temperature development for the lower flange, the web and the upper flange. The non linear temperature distribution in the ribs and the deck obtained with the thermal response models were simplified to linear temperature distributions over the beam and shell elements. For that purpose the average temperature was taken equal to the average temperature over the symmetry line of the thermal response models, see Figure 3, and the thermal gradient was derived such that the temperature of the reinforcement in the structural model equaled the temperature of the node in the thermal response model at the location of the rebars. No failure occurred during the entire fire duration. In the cooling phase, the deflections reduced. See Figures 4 and 5

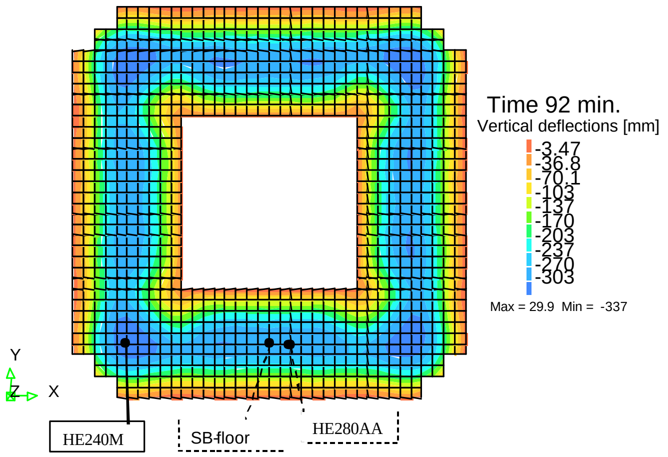

Figure 4: Vertical displacements of the floor.

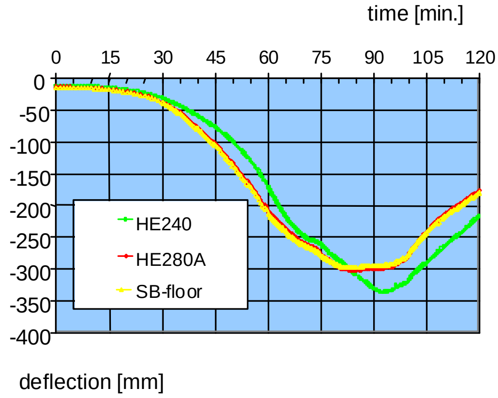

Fig. 5: Deflections of floor and beams.

In addition to the above, the results of the advanced calculations show that – as a result of redistribution of the loads - significant strains occur in the concrete at the corner of the slab (near beams HE240, see Figure 2). This area might be critical (crushing of concrete). Obviously, such a phenomenon is not revealed by an analysis on component level.

Finally one should note that by deleting the fire insulation on the floor beams, a large reduction of the insulation costs compared to the standard fire design was obtained, estimated at some 540 k€ (in 2001).

6 CONCLUSIONS

The alternative FSE design approach gives rise to the following conclusions:

- The functional requirement is met, also if the steel beams are not fire insulated (the columns, however, have to be fire insulated)
- Complementary measures might be necessary (additional reinforcement at the corners of the floor slab).
- Significant cost reduction, compared to the actual solution (i.e. based on the traditional fire safety design)

REFERENCES

[1] Steenbakkers, P. (2001) Brandveilig Ontwerpen van Hoogbouwconstructies, Deel I – Verkennend onderzoek, Graduation report TU Delft (in Dutch)

[2] Steenbakkers, P. (2001) Brandveilig Ontwerpen van Hoogbouwconstructies, Deel II - Case Studie Rembrandttoren, Graduation report TU Delft (in Dutch)

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