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Analysis of Roissy Airport Terminal 2E collapse using deterministic and reliability assessments

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ABSTRACT

Few months after its inauguration in June 2003, a part of the Roissy Airport Terminal 2E in France collapsed in May 2004. The accident was caused by the failure and the drop of the horizontal beam out of its supports and the punching of the concrete vault by the stays used to support it. We study, in this paper, this problem from the available data in order to examine the real reasons of the incident and to see if it was possible to predict the structure failure from the beginning. Our approach is mainly based on comparing the expected results by the design office to the observed values. A deterministic analysis and a mechanical reliability assessment are performed. We show the importance of reliability assessment and long term strains of materials, especially for public constructions where the human and economic repercussions are heavy to bear.

1. Introduction

Inaugurated in June 2003, a part of the Roissy Airport Terminal 2E in France collapsed on Sunday the 23rd of May 2004 causing four dead and three injured. This dire accident, which happened at 7 o'clock in the morning when there was almost nobody on site, could have caused much more casualties if it did happen at a rush hour. The terminal was evacuated and closed to all visitors including investigators, because additional cracks appeared on the next day. The terminal had a length of 650 m and it coasted more than 750 million Euros; its failure had an important economic impact and it has surely affected the image of the French civil engineering.

The accident was caused by the failure and the drop of the horizontal beam out of its supports and the punching of the concrete vault by the stays (compressed metallic bars) used to support it. Yet, no official investigation report was rendered. Some reliability specialists, interested in the accident have already pointed out the low resistance of the structure (insufficiency of steel, low thickness of the concrete ...) and the huge creep deformations. In this paper, we study this problem from the available data in order to examine the real reasons of the incident and to see if it was possible to predict the structure failure from the beginning. Our line of reasoning is mainly based on the comparison of the expected results by the design office to the observed values. Some code-based calculations are made to provide some milestones against which one can contrast observed and calculated values to the prescribed values by the codes standards which were in use during the design of the terminal.

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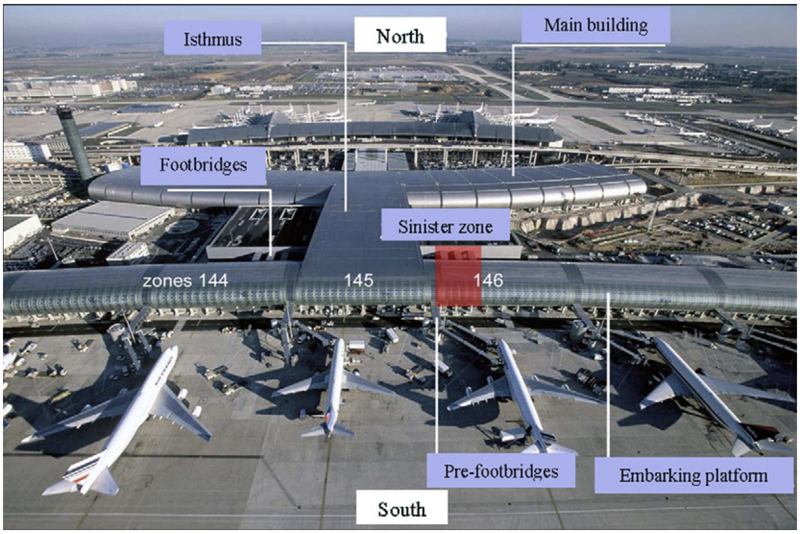

Fig. 1. General view of the Terminal 2E.

In Section 2, a description of the Terminal 2E and its collapse is provided. In Section 3, we draw on a finite elements model of the structure using the ST1 software [L1] for structure calculation. This software enables us to take into consideration the long term deformation of materials (creep and shrinkage) and to simulate the elastomeric bearings by means of a complete[1] stiffness matrix [10–12]. It has to be noted that the edifice is highly complex which requires a fine and complicated simulation model, in order to get valid results. In Section 4, a deterministic sensitivity analysis is performed in order to study the influence of some potentially relevant parameters on the structure (such as the thermal effects, the openings in the horizontal beams, the keying, the creep and the shrinkage of concrete) and to investigate the real reasons of the failure (punching shear and rotation of the bearings) taking into account the long term effects of materials. In Section 5, a mechanical reliability assessment of the structure is performed using a coupling procedure between ST1 [L1] and Phimeca software [L2] which is a reliability software. Furthermore, we study the effect of the existence of additional tensional members between the arcs ends with a better compressive strength of concrete for the whole structure. In Section 6, an overall analysis is performed and possible solutions are proposed.

2. Terminal 2E

The Terminal 2E is made of three parts: the main building, the embarking platform and the isthmus which connects these two buildings (Fig. 1). The design of zone 144 is perfectly identical to the collapsed zone 146.

The shell of zone 146 is made of juxtaposed full arcs and perforated arcs (Fig. 2).

Each arc is made of three segments linked together by keyings. The metallic combined system of tensional members and stays aims at reducing the deformation of the arcs. The arcs lay on horizontal beams which are perforated by ventilation ducts. The horizontal beams are supported by columns by means of elastomeric bearings.

2.1. Collapse of the terminal

On Sunday 23rd of May, at 6:57, a part of zone 146 of the embarking platform (six arcs located next to the isthmus) suddenly crashed (Fig. 3).

The examination of the sinister zone let us make the following observations:

- The horizontal beam slipped out of the supports and was broken right at the pathway of a ventilation duct.
- The elastomeric bearing support located at the extreme west is still in place. Concrete is intact. The support of the middle column is gone. Concrete is a little smashed. The support located at the extreme east (isthmus side) is toppled over. Concrete is smashed.
- The shell is shear punched by some stays on the extreme full arc (isthmus side).

[1] By complete stiffness matrix, we designate a stiffness matrix for a system before taking into account the restrained degrees of freedom due to the supports (limits conditions).

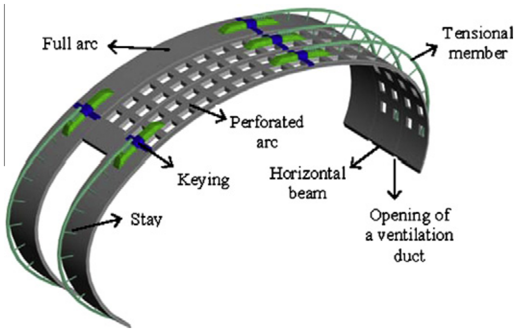

Fig. 2. Geometry of two perforated arcs and one adjacent full arc.

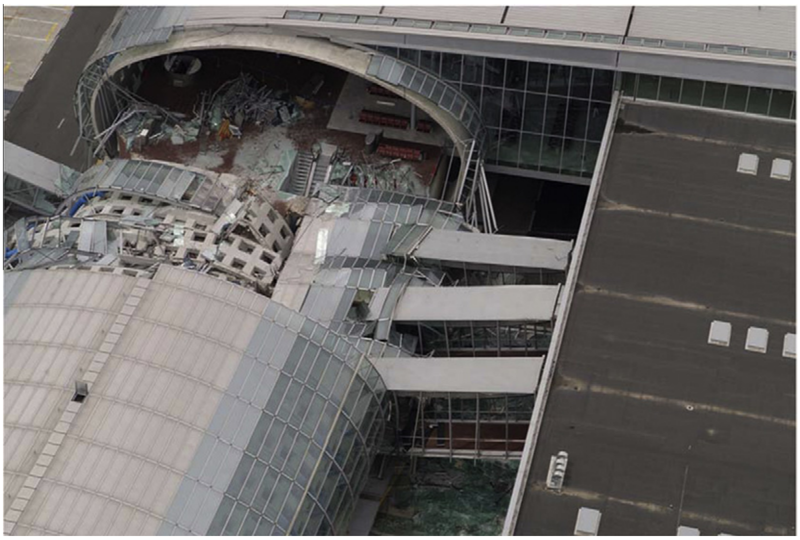

Fig. 3. Crashing of a part of THE ZONE 146.

3. The model

Our structure was simulated on ST1 software [L1] by a bar model that required 1558 nodes and 2320 bars (Fig. 4).

It has to be noted here that we have taken into consideration the complexity of our structure [4] ((i) openings in the shell, (ii) keyings between the segments, (iii) combined system of tensional members, stays and concrete, (iv) irregular cross section of the horizontal beams [L3], (v) openings in the horizontal beams due to the crossing of ventilation ducts, (vi) connection between the arcs by fragile corners iron, (vii) shortened arcs right on the footbridges placement, (viii) simulation difficulties of the elastomeric bearings by means of a complete stiffness matrix [11,12] and (ix) finally the dissymmetry of the structure and the applied loads).

This model is modified subsequently in order to study the influence of different potentially relevant parameters on the construction, taking into account the long term effects of materials (creep and shrinkage) [5,6].

3.1. Results of ST1 model and comparisons

The obtained results have been summarized in Table 1.

The expression «Short term» means right after the arcs adjustment phase (63 days after the casting) [9,10]. The expression «Long term» means at the moment of the collapse (915 days after the casting) [7,8]. The expression «Measured» means measured on site. The expression «Predicted» means predicted by the design office before construction.

The empty cells in the "maximum rotation" columns which correspond to the critical values of the rotation for the long term (corresponding to the failure) were not measured.

After the examination of the abovementioned results, we can make the following observations:

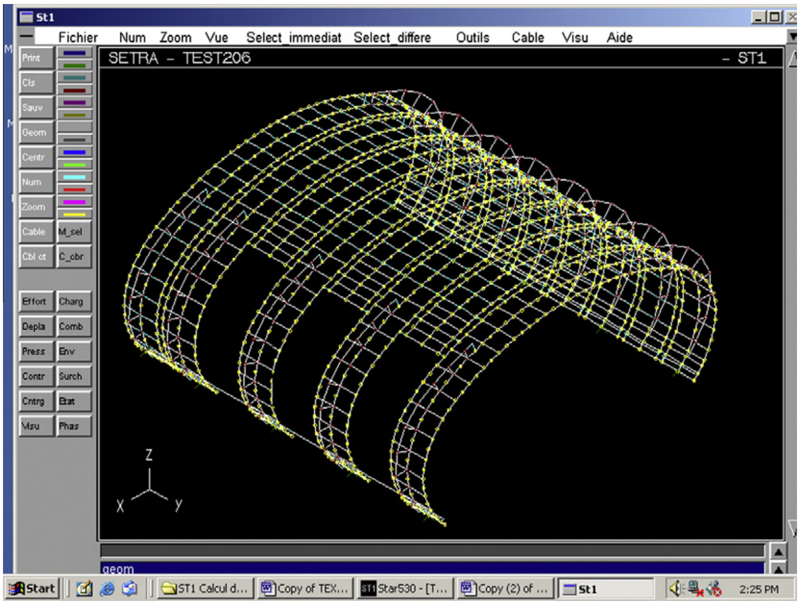

Fig. 4. Model of the first eight arcs starting from the isthmus.

Table 1
Results of ST1 model and comparisons.

Case	Maximum vertical deflection (cm)	Maximum horizontal deflection (cm)	Maximum rotation (rad)	Maximum rotation (°)
ST1 model (short term)	11.74	6.63	0.183e−1	1.05
ST1 model (long term)	18.58	10.56	0.348e−1	1.99
Relative variation	58%	59%	90%	90%
Measured (short term)	11	5	0.244e−1	1.4
Measured (long term)	20	12	–	–
Predicted by the design office (short term)	5	–	0.087e−1	0.5
Predicted by the design office (long term)	Not calculated	Not calculated	Not calculated	Not calculated

- We notice a relative increase of deflection between the short term state and the long term state. This increase can reach up to 59% for the displacements and 90% for the rotations of the elastomeric bearings; this shows the big difference between both states due to the long term effects of materials [5,6] and the importance of taking these effects into account especially when it comes to public constructions that have a complex design.
- There is conformity between the values observed or measured on site and the values obtained by ST1 model [L1] calculation, which confirms the validity of our model.
- The deviation is important between the values observed or measured on site and the values calculated before the construction of the edifice (predicted); this reveals the inadequate modeling that was used by the design office.
- The maximum rotation of the bearings for the short and the long terms, as given by the ST1 model are 0.0183 rad and 0.0348 rad respectively. Whereas, the elastomeric bearings have an allowable angle of rotation of $\alpha_{all} = 0.0167$ rad [11,12]. Moreover, although the results predicted by our model regarding the rotations of the bearings are closer to the reality than those predicted by the design office, one can notice that there is still a difference of about 33% between our values and reality. In fact, we have shown in previous studies [3–17] that the creep of the concrete shell, which is one factor among others that increase the deflection of the shell and the rotations, is underestimated by formulas suggested by almost all the design codes.

This shows the accuracy of our calculation which has given results relatively close to the reality, contrary to the calculation done before the construction by the design office.

4. Deterministic influences of some parameters

4.1. Sensitivity analysis

The aim of this part of the study is to assess the effects of neglecting the following factors:

- The variation of temperature [10], given that there was a sudden decrease of the temperature a few days before the sinister.
- The openings in the horizontal beams due to the crossing of the ventilation ducts.
- The concrete used for the keyings, since there was a doubt on the concrete quality.
- The creep of concrete [5].
- The shrinkage of concrete [6].

We summarize in Table 2 the maximum relative variations of deformations due to the different considered parameters: We notice the following:

- The effect of creep is the most important among the studied factors. The creep deformations have reached a relative increase of 55.5% in 1 year approximately.
- The shrinkage and thermal effects had little influence; hence we can say that the variation of temperature which preceded the sinister was not one of the main causes of the edifice collapse.
- Other parameters had practically no influence.

4.2. Deterministic justification of the collapse

4.2.1. Punching shear of the shell by the stays

The maximum compression forces in the stays at the serviceability limit state, after incrementing according to the Swiss Standard SIA 162 in order to take into account the non-uniform distribution of shear stress [1], are (forces are calculated in tons):

$$\begin{cases} V_d(\text{Short term}) = 125.37T \\ V_d(\text{Long term}) = 149.44T \end{cases}$$

The ultimate punching shear strength of the shell (using the BAEL which is the French design code [2] or the SIA 162) is equal to:

$$V_{Ru} = 73.275T$$

According to the Swiss Standard SIA 162 [1], and after taking into account the presence of steel bars, the punching strength can be increased up to a maximum of 50%. Moreover, a compressed shell will have an additional strength of 30% to punching; hence, the ultimate strength of the shell is:

$$V_{Ru} = 73.275 \times 1.5 \times 1.3 = 142.89T$$

By comparing the obtained forces V_d to the ultimate strength, we notice that:

- In the short term, $V_d(\text{Short term}) = 125.37T < V_{Ru} = 142.89T$ (the limit state is not reached).
- In the long term, $V_d(\text{Long term}) = 149.44T > V_{Ru} = 142.89T$; then, there is an exceeding of the ultimate strength.

This explains the fact that the structure did not fail at the short term, while at the long term the force exceeded the resistance, which caused the punching of the shell by some stays, and triggered hence the ruin of the edifice.

4.2.2. Rotation of the bearings

As mentioned in Section 3, the maximum rotation of the bearings for the short term and the long term, as given by the ST1 model are 0.0183 rad and 0.0348 rad respectively. Whereas the elastomeric bearings have an allowable angle of rotation of $\alpha_{all} = 0.0167$ rad [11,12].

Table 2
Maximum relative variations of deformations.

Parameter	Temperature	Openings in the horizontal beams	Concrete of keyings	Creep	Shrinkage
Relative variation of the deformation	7.5%	0.11%	1%	55.5%	5.3%

Table 3
Random variables of the strcuture [3–16].

Parameter	Distribution law	Mean values	Standard deviation
Compressive strength, f_{c28}	Lognormal	40 MPa	4 MPa
Density ρ of concrete	Normal	2.5 T/m^3	0.08 T/m^3
Hygrometry, ρ_h	Gumbel	70%	5%

We notice that the rotation of the bearings is unacceptable at the short term and at the long term as well, and that the rotation at the long term is more than double of what is allowable. This reveals the initial deficiency of the structure and explains the toppling over of some supports.

5. Reliability-based assessment of the structure

In this section, we apply a reliability-based assessment to the structure [13,14,18]. The mechanical-reliability coupling is carried out between the software ST1 [L1] and Phimeca [L2]. This coupling procedure is often the only way to compute the reliability of real structures, where the mechanical behavior is given by finite element analysis. At the beginning of the procedure, Phimeca prepares the input file of the studied structure to be analyzed by ST1. An iterative scheme is then applied, where Phimeca performs a number of calls to the finite element software ST1 which produces the creep deformations for a given realization of random variables. When convergence is reached, Phimeca evaluates the failure probability and the sensitivity measures. This coupling procedure, mandatory for our study, necessitated a large amount of work to automate the dialog protocol between the reliability and the mechanical softwares.

Previous deterministic and reliability studies [15,16] have shown that creep is primarily sensitive to three parameters: the compressive strength of concrete f_c, the hygrometry ρ_h and the density ρ of concrete. To take account for uncertainties, these variables are considered as random, while the other parameters (such as the mean radius r_m, the steel quantity ρ_s, and the loading date t_0) are taken deterministic because their influences are negligible. Table 3 gives the model variables with the corresponding probability distributions and their first and second moments (i.e. mean and standard deviation). The data used have been confirmed by engineering expertise [SETRA Service d'Etudes Techniques des Routes et Autoroutes – France and LCPC Laboratoire Central des Ponts et Chaussées – Paris].

For this structure, verifying the limit state consists in checking whether the short term creep deflection is above the permissible level which is calculated, since we do not have architectural elements that are likely to be damaged, as following [1]:

$$f_{all} = \frac{l}{250} = \frac{31}{250} = 0.124\ \text{m} = 12.4\ \text{cm} \quad (\text{where } l \text{ is the span length})$$

We have to mention here that the role of an excessive deflection is to lower the shell in the middle, to increase the horizontal displacement of the supports and to induce the rotations of the bearings.

So the failure probability P_f is given by [13,15,18]:

$$P_f = prob[(12.4\ \text{cm} - f) \leqslant 0]$$

In First Order Reliability Methods, *FORM*, the reliability index β can be simply defined by [13]:

$$\beta = -\Phi^{-1}(P_f)$$

where $\Phi(\cdot)$ is the standard Gaussian cumulative distribution function.

Our calculations resulted in a reliability index $\beta = 1.8244$ and a probability of failure $P_f = 0.0340$.

This probability of failure is too high and unacceptable, compared to the values given by Calgaro [15] and which are in the range of 10^{-5} and 10^{-3}.

While the deterministic calculation of the deflection did not present any deficiency at the short term, the reliability study has shown that the structure presented an unacceptable (according to all the design codes) high probability of failure at the short term also.

Whereas at the long term, the results of the reliability study corroborate those obtained by the deterministic study which indicated that the deterministic deflection is bigger than the allowable one.

6. Analysis and possible solutions

The collapse of the terminal had an important impact on economy and tourism in France. Thus, we are interested in probing the possibility of repairing the terminal, and in determining if the introduction of structural reinforcements in the initial design would have prevented the catastrophe.

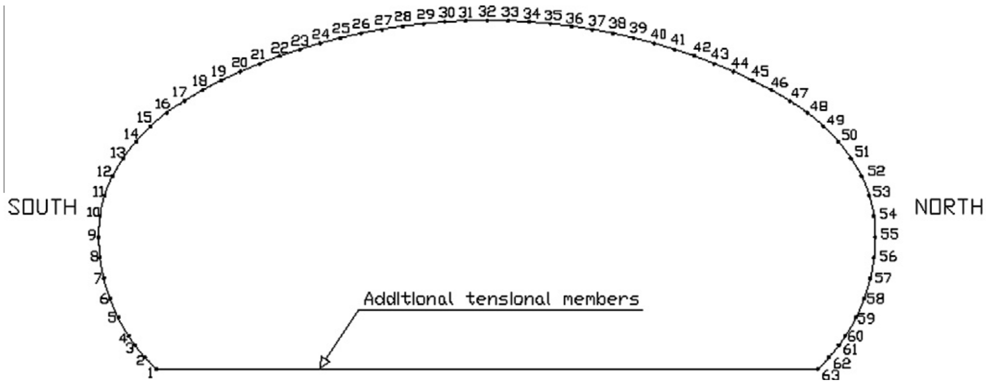

Fig. 5. Additional tensional members linking both ends of the arcs.

First of all, the mending of the structure, after all the deformations that it has sustained, would be too expensive. Moreover, we shall not forget that the public might not be convinced of using the terminal anymore, especially after all the media fuss on this subject.

On the other hand, we have clearly demonstrated the structure deficiency at the short and long terms. So, we conclude that the design is in itself bad and not well-conceived. Therefore, it would be better to entirely change it from the outset. However if we want to keep the same architecture, we should opt for the reinforcement alternative. For this reason, we have studied possible modifications making the structure "stronger." Two solutions have been considered:

- Using tensional members that link the end of the arcs at the bottom level (Fig. 5).
- Improving the compressive strength of concrete by taking f_{c28} = 55 MPa instead of 40 MPa.

We have to mention here that the role of the tensional members that link the end of the arcs at the bottom level is to reduce the horizontal displacement of the beams which contributed to the rotation of the bearings, and hence, to the collapse mechanism. Moreover, it reduces the elastic part of the vertical deflection of the shell.

The obtained results for the compressive forces in the stays at the serviceability limit state are:

$$\begin{cases} V_d(\text{Short term}) = 119.23T \\ V_d(\text{Long term}) = 141.61T \end{cases}$$

Since f_{c28} has increased, the ultimate punching shear strength of the shell will also increase. Hence, we get:

$$V_{Ru} = 166.28T$$

By comparing the forces at the serviceability limit state at long term with the ultimate strength, we notice that V_d(Long term) = 141.61 < V_{Ru} = 166.28, and so, the safety factor will be:

$$\gamma_R = \frac{V_R}{V_d} = \frac{166.28}{141.61} = 1.17$$

The obtained safety factor is obviously not sufficient. It is not our intention in the framework of this paper to suggest an alternative new design for the structure. The sole purpose of this paragraph is to give some perspective to the collapse mechanism and how to improve the stability and strength of the structure.

7. Conclusion

The obtained results have revealed the inadequate modeling that was made by the design office and the primal influence of the creep of concrete at the long term, which caused excessive deformations unpredicted by the design office before construction for two reasons: (i) A rough simulation of the structure; (ii) not taking into account the concrete deformations at the long term (especially creep deformation).

Our study has shown that the long term deformations have produced excessive forces in the stays which led to the shear punching of the shell. They caused also important rotations of the elastomeric bearings exceeding the allowable thresholds. Moreover, we have performed a reliability assessment of the structure. The result was not surprising: we got a high probability of exceeding the allowable deflection even at the short term; this result confirms certainly the deterministic study.

However, we showed that the presence of supplementary tensional members between the arcs ends and that a better compressive strength of concrete could have prevented the sinister.

Our study leads us to conclude that the design office failed to: (i) Perform calculations taking into consideration the long term effects of materials (creep, shrinkage, relaxation, ...); (ii) make sufficiently detailed models of the structures in order to be faithful to reality, especially when it concerns non-classic edifices from the structural or architectural point of view; (iii) assess the reliability of constructions especially when the failure causes serious consequences.

Finally, it has to be noted that the keyings between the segments were also broken and it would be interesting to make a plastic nonlinear calculation of these zones to see whichever happened before the other, the punching of the shell by the stays or the failure of the keyings.

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