Learning from failure of a long curved veneer wall:

Structural analysis and repair

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Abstract

Masonry remains the most used material for enclosure and partition walls. Still, cracking often occurs in unreinforced masonry walls under service conditions and scarce information can be found about systemic approaches in real case studies. Here, attention is given to a large building faced with fired clay brick masonry veneer that exhibited significant out-of-plane movements only 1.5 years after construction. During the operation of dismounting part of the veneer, partial collapse occurred. The veneer wall, with an elliptical shape, 240 m length and 15 m height, presented no movement joints and insufficient tying. Irreversible expansion of the fired clay brick masonry was expected to be the main reason for damage. The combination of inspection works (visual inspection, pull-out tests and topographic survey) and advanced structural analysis with finite elements, allowed to understand the damage and propose a repair solution at a fraction of the cost of demolishing and rebuilding a new veneer wall.

Key-Words

Masonry veneer; Irreversible brick expansion; Structural analysis; Repair and strengthening

Introduction

Masonry is possibly the most used material for enclosure and partition walls, independently from the structural solution adopted for the building. In Portugal, masonry walls, including finishings, accounted typically for about 15% of the building cost, Bezelga (1984). Masonry walls are also responsible for 25% of the defects in buildings, Bureau Securitas (1984). From the damage in enclosure walls, around 50% are due to cracking and water leakage, both for France and Spain, Bureau Securitas (1984) and ASEMAS (1997). The performance of (non-structural) masonry walls is usually linked to the structural system and the foundations selected for the building. In particular, it is normal that damage results from inadequate behavior of the beams, slabs and foundations, due to shrinkage, creep, thermal movements, excessive deformation and soil settlements.

The financial risk of masonry walls is demonstrated by the Spanish experience, ASEMAS (1997), where 20% of the damage claims are accepted by common agreement and the legal actions are judged favorable to the petitioner in 70% of the cases. The designer (the architect in Spain) is considered the sole responsible in 12% of the cases and co-responsible together with the other agents in the remaining 88% cases.

Design data is available in the codes together with rules of good practice, often with a semi-empirical basis, for example for the definition of movement joints and minimum percentage of reinforcement for crack control, EN 1996-1-1:2005 (2005). Still, not much consolidated research can be found about systemic approaches towards masonry damage and failure in real case studies. Examples include Van Zijl *et al.* (2004), which address the aspect of walls under restrained shrinkage and movement joints in the Netherlands, Fathy *et al.* (2009), which address the solutions and cracks in Spanish veneers, Dilrukshi, *et al.* (2010), which address the propagation of cracks in

masonry walls under a time varying thermal load in an overlying roof slab, and Del Coz Díaz *et al.* (2011), which made a detailed analysis of the failure of an external clay brick wall built in 1995. Here, attention is given to the failure of a brick veneer wall in Portugal.

The Multipurpose Hall in Gondomar, Northern Portugal, has been completed in June 2007 with a total built area of 12 483 m² and is composed of three blocks: the main building of elliptical shape, the administrative building of rectangular shape and the technical areas and warehouse buildings (separated from the rest). The elliptical building has 6500 seats and an area of 6500 m², with axes of 104.4 m x 84 m and a height between 15.5 m in the façade and 21.4 m in the center, see Figure 1.

The buildings are all faced with fired clay brick masonry veneer, which exhibits significant out-of-plane movements. Due to the risk of collapse, it was decided to dismount and rebuild a localized area of the façade around the south wall (about 15 x 15 m²), where the displacements were larger. During the operation of dismounting, in September 2008, partial collapse of the veneer wall occurred, fortunately without any casualties.

Subsequently, a team from University of Minho was appointed to determine the causes of the observed damage, to carry out the safety assessment of the veneer wall and to define repair solutions, which are addressed in the present paper. Full details can be found in Lourenço *et al.* (2009).

Characterization of the adopted building solutions

The building is located in open field, free from the influence of neighboring buildings. The masonry veneer wall has a perimeter of 242 m and a height of 15 m. The masonry veneer is continuous, with the exception of four large openings and nine small openings

for access, even if the European Norm EN 1996-2:2006 (2006) recommends using movement joints for a maximum length of walls of 12 m.

The transversal section of the system is made (from the outside to the inside), by masonry veneer with 0.115 m thickness, a cavity partly filled with a sprayed polyurethane foam with a design specification thickness of 0.07 m, and a structural backing wall in reinforced concrete with a thickness of 0.25 m, see Figure 2. The movements subsequent to the construction lead to a cavity with a thickness between 0.07 and 0.13 m.

The vertical section along the height of the panel is shown in Figure 3, with a support at the base given by a channel steel profile with 400 mm height and a mid-support made with an angle steel profile of 140 mm. At the top, zinc flashing provides protection against water infiltration. In addition, hammer-set wire zigzag ties were applied with a spacing of about 1.0 m in each direction. The ties have a diameter of 4 mm and a length of 0.30 m, with a minimum anchoring in the concrete of 0.05 m and a hook in the masonry unit or bed joint.

The photographic database allows to identify the construction sequence of the veneer wall, which lasted from September 2006 to May 2007 (9 months). Figure 4 shows that the construction was made with isolated panels between the large façade openings. These panels have been built from the lower support up the full height, without any interruption. Afterwards, only the upper part of the panels around the large openings was made, as a final decision from the architecture regarding the large façade openings had not yet been made. Subsequently, the connection of the elliptical veneer to the rectangular building was made and, finally, the lower part of the panel around the large openings was built.

Zinc capping was added at the top of the wall in the full development of the buildings, meaning the panel can possibly move on top without being noticed or provoking major damage. The zinc cap will provide low restraint to out-of-plane movements.

In situ inspection and testing

Visual inspection

Visual inspection indicated that the existing cracking is localized and has small crack widths, see Figure 5a, concentrating mostly around the openings and the construction joints. In the south corner of the rectangular body, a vertical crack indicates expansion problems, see Figure 5b. Finally, a horizontal crack at the elliptical wall mid-height is associated with the out-of-plane movements observed, see Figure 5c.

The observations of the collapsed area allow to conclude the following, see also Figure 6: (a) the wall ties failed due to pullout from in the concrete wall; (b) at mid-height the out-of-plane movement was about 0.05 m, already outside the steel angle; (c) the spacing of ties is about 1.0m with a square distribution; (d) the zinc capping, placed after the wall full completion, did not exhibit any signs of movement or damage.

Topographic survey

Two surveys of the out-of-plane movements were carried out, in July (before the accident) and October (after the accident) 2008. No previous measurements are available, meaning that the initial (reference) configuration of possible out-of-plumb movements is unknown. It will be assumed that the wall was originally built straight, as no additional information exists.

The first measurements were made only above the north and south openings (see Figure 7), as more damage and larger displacements could be observed in the doors' lintels. Figure 8 shows the out-of-plane displacements measured, with increasing values from the base up to a certain height, followed by an inversion of the profile and lower displacements. The maximum value occurs between $\frac{1}{2}$ and $\frac{3}{4}$ of the height for the south door (where the accident occurred), with a value of about 0.07 m. For the north door, the maximum displacement, occurs between $\frac{1}{4}$ and $\frac{1}{2}$ of the height and reaches a value of about 0.03 m. No restriction due to the mid-height support is found. On the contrary, some restriction is clearly observed at the top of the wall, due to the capping.

After the accident, a second (and more detailed) topographic survey was carried out, with vertical alignments obtained each 4 m and a more dense number of points per alignment, see Figure 9. Some variation is observed due to the irregularity of the surface and difficulties in the measurements. Also the displacements around the south door are all zero, as they correspond to the collapsed area. The new measurements allow to conclude the following: (a) the largest displacements occur above the west and east doors; (b) the displacements measured are mostly in the outwards direction, with the exception of the west connection to the rectangular body, where small inward displacements occur; (c) displacements larger than the insertion length of the wall tie in the concrete wall are found, meaning that these ties are fully inactive; (d) displacements at mid-height similar to the support length at the steel angle are found, meaning that this support is partly inactive and the wall slenderness ratio (length / width) reaches 135; (e) similar maximum displacements have been measured before and after collapse for the north door.

Pull-out load testing

As the wall tie producer did not provide any value for the pull-out strength of the ties inserted in the concrete and the inspection indicated that this is the weakest connection, in situ tests have been made. The pull-out tests were carried out in January 2009 and using new ties, as no original ties could be found in the collapsed area. As several ties were bent in the original application and the value obtained is merely indicative for diagnosis, four new ties have been inserted in the concrete wall with the procedures and personnel used in the original ties placement. Two ties have been inserted normal to the concrete wall and two ties were inserted with a 45° angle with respect to the concrete wall.

Test procedures followed the recommendations of ASTM E488 – 96 (2003), recently updated, see Figure 10. The load was applied normal to the surface and the failure mode was equal in the four ties (pulling out of the tie). A linear force-displacement response is obtained up to a first peak value. After a load drop, one or two additional peaks (with a higher value) occur, followed by a total loss of strength. Longer embedding lengths do not provide a strength increase but provide a ductility increase. For the inclined ties, local spalling of concrete around the hole is observed. It was found that the difference in strength between straight and bent ties is very low. The ultimate force for the ties was 1.6 and 1.4 kN, for the straight ties, and 1.5 and 1.7 kN, for the bent ties. The indicative average strength found is therefore 1.5 kN.

Diagnosis and justification for the observed damage

In the previous section, it was stated that the veneer suffered significant outward displacements after construction and that the level of cracking of the wall was low, meaning that the main cause of damage is the irreversible expansion of clay brick. It is

possible that the temperature can also have some contribution to the damage, because the largest displacements are found around the south and east doors. Note also that the construction took place mostly from September to May, so the wall was built in colder months, increasing the temperature effects. A finite element model will be prepared in order to justify the damage and, later, for defining a repair solution.

Adopted finite element models and material data

For the discussion, two finite element models have been prepared, considering the wall ties active (Model 1) or inactive (Model 2). The elliptical masonry veneer is fully represented in the model, above the channel steel profile and up to the connection to the rectangular body, and all openings are included. Model 1 also considers the wall ties, rigidly supported in the concrete wall. All models have been prepared using the general purpose finite element code DIANA® (TNO Diana, 2009).

The masonry panel is restrained at the base and at the connection to the rectangular body. In order to allow sliding of the wall at the lintels, mid-height support and wall top, springs have been considered with a vertical stiffness of 15 N/mm^3 and an horizontal stiffness of half of the vertical stiffness. These values are about $\frac{1}{3}$ to $\frac{1}{2}$ of the experimental values for masonry, see Lourenço and Rots (1997), taking into consideration that the contact surface at mid-height is half of the width of the wall and no information exists about the stiffness of the connection at the wall top.

The structural model considers the following finite elements: (a) three-dimensional bar elements with two nodes to represent the wall ties; (b) shell elements with eight nodes to represent the wall veneer; (c) interface elements with six nodes to represent the supports with possible translation. The material data for the masonry wall is obtained from EN 1996-1-1:2005 (2005), knowing that the compressive strength of

the masonry units is 20 N/mm^2 and the compressive strength of the adopted mortar is 5 N/mm^2 . Then, the characteristic compressive strength of masonry f_k is equal to 5.5 N/mm^2 and the Young's modulus of masonry is given by $1000f_k$ (note that half of this value is assumed for long term actions, such as irreversible masonry expansion). The adopted value for the steel Young's modulus was $200 \times 10^3 \text{ N/mm}^2$.

The α value for the coefficient of thermal expansion of masonry is in the range of 4 and $8 \times 10^{-6} \text{ }^\circ\text{K}^{-1}$, according to EN 1996-1-1:2005 (2005). In the absence of more information, a value of $6 \times 10^{-6} \text{ }^\circ\text{K}^{-1}$ is used. The thermal resistance coefficient R_{mas} has been calculated from the thermal conductivity of the masonry unit, as provided by the producer Ceramica la Oliva, Spain, (0.33 W/m.K), and of the mortar, from literature, resulting in a value of $0.207 \text{ m}^2.\text{K/W}$.

EN 1996-1-1:2005 (2005) does not provide enough information to quantify the irreversible expansion S_w , as this value depends significantly on the clay used for the masonry units, and the recommend value is anywhere between -0.2 e $+1.0 \text{ mm/m}$. An analytical model is available in the literature, Brooks (1990), so that the expansion coefficient can be obtained from the characteristics of masonry components, namely elasticity modulus, expansion / shrinkage coefficients and geometry.

The expansion value provided by the masonry unit producer is 0.5 mm/m . This value is obtained using EN 772-19:2000 (2000) by boiling the material for 24 hours and, according to Menezes *et al.* (2006), this is equivalent to the naturally occurring expansion in 36 months. The expansion of clay units is an irreversible phenomenon that continues indefinitely and can be approximated by a power law for ages larger than 10 days, Wilson *et al.* (2003). The calibration of this law can be made from the value given by the producer, leading to the evolution law for the masonry units' expansion given by Table 1 up to the expected service life of the building (50 years).

It is shown that the expansion doubles from 36 months to 50 years. From the building sequence discussed above, the closing of the elliptical veneer is made approximately 210 days after starting the construction, and the reference time considered will be therefore 105 days (or half of this period). For the date in which the topographic survey was made (17 months), only 19% of the expected full expansion had occurred, meaning that the displacements would be magnified by five, in the absence of any repair action.

Using these results in combination with Brooks (1990) formulas, slightly lower values are found for the expansion of the masonry due to the beneficial effect of the mortar. Moreover, different values are found for the vertical and horizontal direction due to the staggering of the masonry units. The final values obtained for the masonry are, in mm/m, $S_{wx}(17 \text{ months}) = 0.131$ and $S_{wx}(50 \text{ years}) = 0.703$ for the horizontal direction and $S_{wy}(17 \text{ months}) = 0.119$ and $S_{wy}(50 \text{ years}) = 0.641$ for the vertical direction.

Actions

The actions considered in the structural analysis are the self-weight, irreversible expansion, temperature and wind. For the self-weight, a value of 18 kN/m^3 has been considered. The irreversible expansion, taking into consideration the expansion values obtained before and the thermal expansion coefficient, can be simulated by a uniform temperature change of about 20° C for the 17 months period and about 90° C for the 50 years period.

The definition of the temperature load is complex, see EN 1991-1-5:2003 (2003), including three components, namely uniform temperature variation T_u , variable temperature along the length of the element T_{My} and variable temperature along the

width of the element T_{Mz} . T_u provides a volumetric expansion of the structural elements, T_{My} provides an additional uniform value that changes along the envelope of the building according to the orientation and T_{Mx} considers the external and internal temperature of the building, together with the thermal resistance. Figure 11 shows the adopted distribution of temperature uniform and temperature gradients calculated for the different conditions of solar orientation and seasons.

The definition of the wind load has been made according to EN 1991-1-4:2005 (2005). The wind load W is given by the dynamic pressure q_p multiplied by a coefficient c_{pe} . The value q_p varies almost linearly along the height between 0.5 and 1.0 kN/m² and the value of c_{pe} is variable, being given in Figure 12.

Results of numerical analysis

The results for Model 1 (i.e. with the ties) are not shown here, but it was found that the response was totally inadequate as the force in the walls ties was, in many cases, above the measured strength of 1.5 kN (reaching up to 5 kN), the maximum out-of-plane displacement was about 2 mm, which does not replicate the values in the range of 50 mm observed, and the variation of displacements along the height was also not according to the values measured. The obvious conclusion is that the ties are currently inactive and Model 2 (i.e. without the ties) will be used for further comparison.

Figure 13 shows the displacement field for the actions with larger values, namely the masonry expansion and the uniform temperature variation. Figure 14 shows the values obtained and measured for each door (again the south measurements are not present due to the collapse observed). Here, “Exp.” means irreversible masonry expansion, “Temp.” means temperature, “Temp. U.” means the uniform temperature oriented to south, “Temp. G.” means the gradient temperature oriented to south and

“Wind W” means wind oriented to west. It can be observed that the largest contributions to the deformation are given by the masonry expansion and the uniform temperature. The combination of temperature and expansion provide maximum values and a distribution of displacements along the height in reasonable agreement with the measured values. Still, it is noted that the irreversible expansion suffered is only 19% of the final value for the building service life, meaning that the most relevant action is the long term irreversible masonry expansion. Finally, Figure 15 shows the distribution of tensile principal stress due to masonry expansion, which is similar to the distribution obtained with the uniform temperature variation (also in terms of maximum values). It can be seen that the maximum value only for this load is about 0.25 N/mm^2 , leading to the horizontal crack that can be observed in the structure at the mid-height support.

Finally, it is noted that a non-linear analysis was not carried out for this case study. In the authors’ opinion the selection of a structural analysis tool and its features needs to consider the benefit of the information obtained, as well as the cost and the time requirements. Even if this would be relatively easy to carry out with the adopted finite element code, this was considered not required to justify the observed damage or to assess the safety of the proposed intervention measures. This contrasts with many other applications, particularly to historic buildings or unreinforced masonry under seismic loading, in which the authors consider the need to use non-linear analysis in engineering applications, e.g. Lourenço *et al.* (2011) and Marques and Lourenço (2011).

Repair or rebuild?

It was found that: (a) the number of ties in the structure are clearly insufficient to resist the actions occurring in the structure, and much lower than the producer recommendations (1 per m^2 instead of 5 per m^2); (b) irreversible masonry expansion and

temperature lead to high stresses and large displacements of the veneer wall, after the ties became inactive; (c) irreversible masonry expansion will continue, reaching at the planned service life (50 years) a value five times higher than the actual value. It is certain that the observed problems would not exist (or would be significantly less severe) if movement joints had been originally considered. Other provisions to reduce the masonry expansion were not considered, namely by ensuring a significant period of time between brick production and brick use and by using a very poor mortar capable of accommodating some of the expansion. In the current condition, it is certain that repair or rebuild of the veneer is needed.

The following aspects will therefore be considered in the discussion of defining adequate repair solutions: (a) the effect of the most relevant actions can be significantly reduced by adding movement joints; (b) the wind action is not much relevant in the case of veneer without movement joints, but should not be neglected in the design of the repair solution; (c) given the significant movements observed, second order effects should not be neglected in the design of the repair solution; (d) the restraint provided by the mid-height support and top support seems to be weak, and can be ignored for the design of the repair solution; (e) given the large displacements observed most ties are inactive, and should be ignored for the design of the repair solution.

Description of possible solutions

Three alternative solutions have been considered for a cost analysis, namely by placing new ties from the outside, incorporating movement joints or demolishing and rebuilding the wall, as described next.

The first possible solution considers only new wall ties, with higher density and / or higher strength. Such a solution does not allow for any possible stress relief from

previous expansion (if at all possible). The proposed solution is to use a chemically fixed threaded bar inside a steel sleeve, as shown in Figure 16. As shown, the ties are inserted in the cross joint of the masonry, so that no damage occurs in the masonry brick and no additional treatment of the hole is necessary, as the epoxy color is similar to the mortar joint color. In the vicinity of the openings, mid-height support and top support, the number of ties is doubled due to the foreseen stress concentrations.

EN 1996-2:2006 (2006) recommends using movement joints and, in this case, thermal variations and irreversible masonry expansion do not have to be considered in the design. In addition to the creation of movement joints by sawing, wall ties similar to the ones just described need also to be added to the veneer, but with a lower density. For clay brick masonry, the maximum recommended spacing between vertical joints is 12 m. According to the possible modularity this value cannot be strictly considered and the proposed placement of straight vertical joints is indicated in Figure 17. No recommendations are given for the width of the joint and the sealing material. The joint width has been calculated according to MDG-5 (2007), leading to a value of 25 mm.

The spacing of horizontal joints to accommodate vertical displacements is not addressed in EN 1996-2:2006 (2006). This also brings difficulties due to the self-weight of the wall and, in the present case, the very low bending stiffness of the ties could lead to additional cracking of the wall due to the vertical movements arising. Therefore, no horizontal joints are planned.

Finally, the third solution includes the full demolition of the veneer wall, followed by reconstruction taking into account movement joints and an adequate number of traditional wall ties.

Safety assessment and cost estimate

Adding ties to the finite element model (Model 2) described before, it is possible to make the calculation of the forces in the ties for the load combinations involving all the necessary actions (self-weight, second-order effects, irreversible expansion, temperature and wind). The safety assessment is made assuming a design strength for the ties of 2 kN and the tensile stresses in the masonry panel are verified. For the solution 1 (only ties added) and 2 and 3 ties per m², the code safety is not verified for the load combination involving temperature. Using 4 ties per m², the expected service life is only 10 years, with failure due to irreversible expansion combined with temperature loading.

In the case of Solution 2 also movement joints are added in the finite element model. Figure 18 shows the envelop for all load combinations and for each tie level, when one tie per m² is used. In general, the solution can be accepted, with a very small area over 2 kN, which can be taken by neighboring ties.

The cost of the three solutions has been estimated according to market prices in Portugal, using as reference the cost for demolition and reconstruction. Solution 1 (addition of four ties per m²) costs 73% of the reference solution, with an estimated service life of 10 years only, being not recommended. Solution 2 (addition of one tie per m² and movement joints) costs 26% of the reference solution, with an estimated service life according to the code. It was therefore recommended to use Solution 2.

Conclusions and lessons learnt

Architects and engineers adopt sometimes higher risk solutions be it for aesthetic reasons, cost reduction, technological challenge, arising of new material and techniques, ignorance or excessive confidence, or other reasons. Masonry enclosures often feature

problems, e.g. cracking and lack of water tightness, due to poor detailing, selection of inadequate solutions or poor execution.

Here, a case study involving significant out-of-plane movements in a very large masonry veneer was presented. The wall presented no movement joints and insufficient tying. From inspection, in situ testing and advanced numerical analysis, it was possible to understand the causes of damage and propose a repair solution at a fraction of the cost of demolishing and rebuilding a new veneer.

The lessons learned from this case study are that: (a) a decision to deviate from normal rules of good practice for the placement of movements joints in masonry façades requires careful studies in the design phase, particularly if the wall is curved; (b) irreversible brick expansion is a continuous process that can reach the double of the values provided by producers for a 50 years' service life; (c) provisions to reduce the masonry expansion can be taken into consideration, namely by ensuring a significant period of time between brick production and brick use, and by using a very poor mortar capable of accommodating some of the expansion; (d) a repair solution adopting ties placed from the outside can be much adequate to reinstate the connection between a damaged veneer wall and a backing wall (in the present case, the cost involved was about $\frac{1}{4}$ of the cost of rebuilding a new veneer wall).

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(a)

(b)

(c)

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(a)

(b)

(c)

Figure 6 – Photos after the accident: (a) collapsed area; (b) detail of a wire tie in the rubble; (c) out-of-plane displacement at mid-height

Figure 7 – Out-of-plane displacement survey before the accident: (a) Vertical alignments for measurements above north and south opening; (b) damage and movements in door posts and lintel (south door).

Figure 8 – Out-of-plane displacements measured before the accident.

(a)

Figure 9 – Out-of-plane displacement survey after the accident: (a) 3D view of the alignments measured (each 4 m); (b) examples of out-plane measurements.

Figure 10 – Test setup for pull-out tests: (a) tie inserted normal to the wall; (b) tie inserted with a 45° angle.

Figure 11 – Variation of the external temperature in the building envelop for the four possible orientations in the summer (west, north, east and south) and for the winter.

Figure 12 – Wind distribution in the building envelope for four different orientations (west, north, east and south).

Figure 13 – Displacement field (model without ties) for: (a) masonry expansion; (b) uniform temperature variation. Values are shown in meters.

Figure 14 – Out of plane displacements (model without ties).

Figure 15 – Main principal stress (tension) due to temperature expansion (in kPa).

Figure 16 – Details of repair solution 1: (a) typical vertical section; (b) view of the tie placing around the mid-height support.

Figure 17 – Repair solution 2: Proposed location of the vertical joints.

Figure 18 – Solution 2: Envelop of tie forces for a given height and considering all load combinations, for one tie per m².

Table 1 – Expansion of masonry units in time (up to expected service life of 50 years).

Time T	10 days	30 days	60 days	90 days	105 days	210 days	12 months	17 months	2 years	3 years	5 years	10 years	20 years	50 years
Expansion (mm/m)	0.15	0.20	0.24	0.27	0.28	0.33	0.38	0.41	0.45	0.50	0.57	0.68	0.80	1.01
Percentage of total	15%	20%	24%	26%	28%	33%	38%	41%	45%	49%	56%	67%	80%	100%
Relative expansion * (mm/m)	n/a	n/a	n/a	n/a	0.00	0.05	0.10	0.14	0.17	0.22	0.29	0.40	0.53	0.73
Percentage of total*	n/a	n/a	n/a	n/a	0%	7%	14%	19%	24%	30%	40%	54%	72%	100%

*for $T_i - T_{105 \text{ days}}$