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The use of aramid fibres in the restoration of the Basilica of St. Francis of Assisi

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Abstract

Aramid fibres are used to strengthen the vaults of the upper nave in the Basilica of St. Francis of Assisi, after the last important and destructive earthquake of Sept.26.97.

These interventions are divided into two subsequent stages: the preliminary urgent intervention, made just few weeks after that earthquake to eliminate the risks, and then the definitive intervention to provide the final safety levels.

In both cases aramid fibres and epoxy resin were used, because of their characteristics, particularly favourable in the restoration of masonry structures.

In the final interventions, aramid fibres are used to build a net of ribs on the vault extrados, to improve the stiffness and the bearing capacity of the vaults that have lost their original curvature. These ribs are made of a central timber core and layers of multidirectional laminates of aramid fibres (aromatic polyamide), glass fibres and epoxy resin.

1 Introduction

The Basilica of St. Francis of Assisi (Fig. 1) was hit by the strong earthquake that struck central Italy in the night of Sept.26.1997, and a second, stronger quake (VIII degree of Mercalli scale) again shook the Basilica at 11:42 a.m. resulting in the collapse of two frescoed vaults of the upper Basilica, and considerable damage to all the vaults and to the tympanum, a masonry wall of cavity construction, at the left transept. Cracks, partial collapses and permanent deformations were also spread throughout the Convent.

Vaults collapse was concentrated in two zones of the nave, adjacent to the facade and to the transept (Fig. 2). The main cause of the collapse was the earthquake by the pressure produced on the vaults due to the large volume of fill accumulated over the springer zones (Fig. 3). In the vaults of Assisi, in addition, the original curvature was progressively reduced due to the permanent deformations produced by the numerous earthquakes occurred throughout the centuries, whilst, at the same time, the fill, consisting of broken tiles and other loose materials accumulated in centuries of roof repairs, increased.

Under seismic actions the situation becomes much worse than in static conditions because the pressure of the fill. In fact the fill, without any cohesion, alternatively acts only on one side, whilst on the other side the fill is detached; what's more the loose fill follows the movement of the vaults opposing their recovery and facilitating therefore increasing permanent deformations.

The failure mechanism of the vaults close to the facade, filmed by Umbria Television, resulted from the progressive loss of curvature of the ribs. Then a "hinge" was produced in the middle, just where there was the maximum resultant of the fill's mass action, and finally the rib collapsed drawing the vault down with it (Fig. 4). A similar mechanism occurred in the zone close to the transept, where the second vault collapsed.

The collapses were concentrated on these specific zones because, being the direction of the seismic action mainly perpendicular to the nave axis, the nave itself behaved globally like a "beam", with restraint at the ends provided by the stiff façade and transept, as it is clearly shown by a global mathematical model (Figs. 5 and 6). The result was that normal and shear stresses were produced there, in addition to the "local stresses" resulting from the weight of the fill and from the own vaults' mass. All the vaults that survived the quakes were in a very precarious situation. Large cracks were distributed both on the intradox and the extrados and the curvature was lost in several zones (Figs. 7 and 8).

The urgent measures taken in the first month after the main earthquake were mainly the removing of the huge load represented by the fill in the springer zones of the vaults, the filling of the cracks (with a salt free mortar to limit possible damage to the frescoes) and the suspending of the vaults from the roof with a system of tie bars. Those bars (having first inserted springs to maintain the force at the design value, independently by thermal effects and minor vibrations), were connected to the vaults using aramid-reinforced plates glued on the vaults themselves (Fig. 8). The cracked ribs were also suspended from the roof.

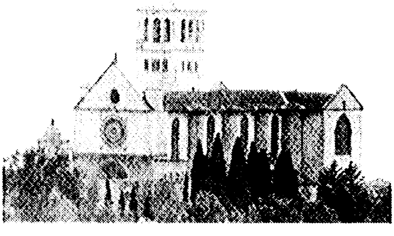


Figure 1: The Basilica.

Figure 2: The collapsed vaults.

2 Seismic actions and mathematical models

The acceleration measured in Sept. 26-97, at the ground close to the Basilica, was about 0,16g in the direction of the longitudinal axis and about 0,18g in the perpendicular direction. Considering an amplification factor of around 1.5÷2, the top of the structure reached about 0.27÷0.36g of transversal acceleration. Many other quakes occurred in the following months, some reaching the VII degree of the Mercalli scale. The earthquake of Sept.26.97, however, has been inferior to those expected by the Italian Code for the Assisi area; that is inferior to the action that we have to take into account in reinforcement design. In fact ground acceleration in order of 0.3g may be foreseen there; moreover, considering also certain amplification factor (about 1.3), the Code expects for usual masonry buildings accelerations in the order of 0.392÷0.4g.

The Basilica and the Monastery are located on a hill stretched in E-W direction and very high and narrow in N-S direction, the direction orthogonal as regards the Basilica's axis. This leads to local amplifications and acceleration of the ground larger than in the middle of the city of Assisi, in case of N-S directed seismic actions (as a matter of fact the Basilica and the Monastery have been more damaged during the last earthquake). We can therefore expect ground accelerations in order of 0.35g in case of action in direction orthogonal to the Basilica's axis.

At the same time it has to be considered that the Basilica structure has a greater amplification capacity of the horizontal actions compared to a normal masonry building (with amplification factor about 1.5÷2 against 1.3).

Moreover we have to take into account the effects of the interventions themselves. The removing of the fill reduces the masses directly acting on the vaults and the riskness of the opposition to recover the deformations during seismic actions, but, on the other hand, allowing higher displacements it allows higher amplifications of the horizontal actions. In fact, after the removing of the fill, the measures taken in 1998 (by ISMES) shows transversal acceleration on the top of the vaults about from 3 to 6 times larger than those on the ground. Naturally in case of greater earthquakes, when parts of the structure come out from the elastic field, there are major energy dispersions and therefore minor amplifications.

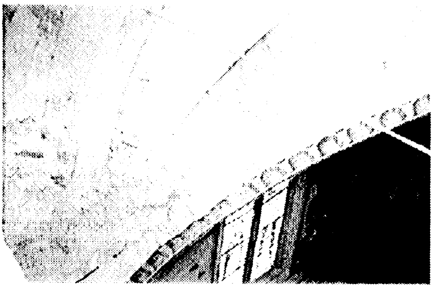


Figure 3: The huge fill.

Figure 4: The collapse mechanism.

At any case, considering also the inevitable stiffening effect of the reinforcement interventions, it is worthwhile to consider amplification in the order of 4 times.

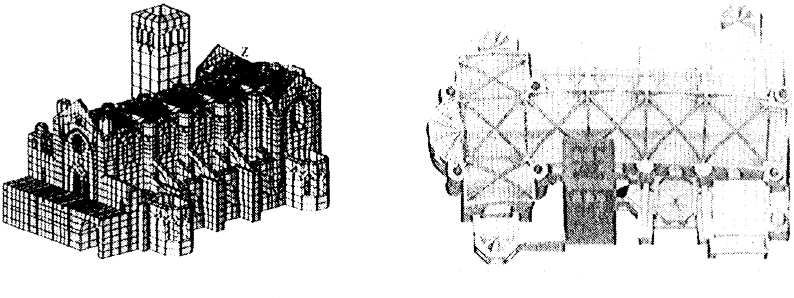
From a different point of view, if a reduced amplification is considered, with a factor in the order of 2÷2.5 with an acceleration equal to 0.7÷0.875g at the top of the vertical structures, it has to be also considered that the vaults, much lighter and less stiff than the supporting structures, may very easily (without the fill) wave out of phase (or in opposite phase) respect to the waving of their springings. That is the vaults may suffer "relative" accelerations, respect to their springings, 1.5÷2 times higher than the "absolute" acceleration applied to the springings themselves.

After all it seems then suitable, for the designed reinforced structure, to consider horizontal acceleration in the order of 1.4g for the vaults mass.

The global mathematical model shows, in case of transversal actions, as the central module of the nave are nearly not affected by side effects, allowing us to study more refined models of only one module.

A first numerical analysis of one nave central module in the situation before Sept. 97 has clearly shown that high tensile stresses (that is cracks) are produced in the ribs and the curvature is reduced also in static conditions (with only the dead loads) because of the huge filling. A preliminary step-by-step analysis with horizontal statically equivalent forces shows that when the seismic action reaches around 0.18g, cracks and permanent deformations are increased, and that at around 0.2÷0.3g may begin the collapse (Figs. 9 and 19). Considering that this is a static equivalent analysis it is clearly explained as all the survived vaults were in a very precarious situation and how the vaults near the façade and near the transept easily received the little additional amount of stresses just necessary to reach the collapse.

A numerical analysis of one vault module in the condition after the earthquake of Sept. 26-97 (using a model reproducing the present permanent deformations without taking into account the accumulated stresses as they are eliminated in the formation of the cracks) shows a clear improvement in the vaults' behaviour after the removing of the fill.



Figures 5 and 6: The global model, the nave's beam-like behaviour. Model made by Ingg. A.Carriero and F.Sabbadini.

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A step-by-step analysis with horizontal statically equivalent forces shows that there is the growing of large deformation when the horizontal seismic action reaches, on the vaults, around 0.4g (instead of 0.18g) and the beginning of the collapse around 0.6÷0.7g (instead of 0.25g) (Figs. 10 and 19); this is, however, a lower action in front of the maximum expected action.

A further numerical step-by-step analysis of the vaults, with horizontal statically equivalent forces in the present situation but with the addition of the designed intervention, shows very reduced deformations with a clear improvement in the vaults' behaviour (Figs. 11 and 19). Moreover the reinforcement structure shows stresses always two times lower then their limit values when the horizontal acceleration is 1.4g (the value considered in their design) (Fig. 12). The mathematical model also shows that a strong partialization and large deformations of the vaults may happen only with more then 1.6g of horizontal acceleration.

3 General criteria to strengthen the vaults

The problem of the definitive restoration and consolidation of the Basilica, especially as regards the vaults, has immediately appeared to be very delicate, because, due to the presence of the frescoes, it was impossible to recover the deformations and to re-establish, therefore, an adequate curvature and autonomous bearing capacity. Different studies, researches and structural analysis have been carried out to decide which solution would have been the best to strengthen the vaults and secure their stability over the time, without producing any risk to damage the frescoes and without compromising the historical value of the original vaults structure. It has been rejected, a priori, the hypothesis to build a reinforced concrete shell or reinforced concrete ribs on the extrados, as too heavy and incompatible with the historical value, or to build steel ribs because of the difficulty to follow the deformed shape of the vaults and, consequently, to obtain a continuous connection among ribs and vaults.

The choice has been to use composite materials to realise, on the vaults extrados, a series of little thin ribs following a pattern typical of Gothic structures (Fig. 13) and letting clearly visible the original structure.

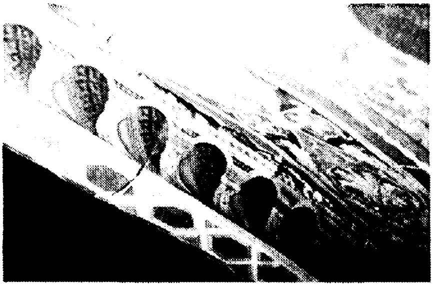

Figure 7: Cracked arch.

Figure 8: Deformed vault shape and provisional suspensions.

Aramid fibres were used, which haven't brittle behaviour, on the contrary to carbon fibres, and present a good ductility. Moreover the ribs are built in situ, so that it is possible to follow the deformed shape of the vaults: whilst the width of the ribs remains constant, the height may be changed in relation with the deformation of the vaults; thus the extrados of the ribs can follow a regular curve parallel to the original ideal surface of the undeformed vaults.

4 Tests

The ribs, made up of aramid fibres bedded in epoxy resins around a central timber nucleus, are light, very strong (the tensile strength of this fibre is 30.000 Kg/cm^2 and that of the fibre with resin is about 14.000 Kg/cm^2) and less stiff of the steel, the elasticity modulus of the fibre being 1.200.000 Kg/cm^2 and that of the fibres with resin 600.000 Kg/cm^2.

Different samples of the constitutive materials and of the composite ribs have been tested in specialised laboratories (Fig. 14), as up-to-day there was no large experience on the application of these products to consolidate historic masonry structures. The tests allowed to refine the ribs design in such a way to improve their mechanical characteristics and, contemporary, to reduce their section to a minimum, that is to reduce the impact of this new technology on the existing structure.

The first idea was to build box shaped ribs in aramid four-directional fibre tissue layers stratified with epoxy resin, similar in shape to those utilised to stiff the hull of the ships. In fact it is possible to consider the extrados of a vault as the reverse of a ship hull. Such box shaped ribs are provided by a base, larger than the rib itself, for the gluing to the vault extrados surface. The connection to the vault is due, besides to the gluing, also through aramid fibre pivots.

The different load conditions, compared to those of ship hull reinforcements, bring to higher moment and shear actions, in case of comparable rib length. Thus to avoid to use too large sections the bending strength has been assured by inserting pultruded flat composite bars in the extrados and in the intradox of each rib.

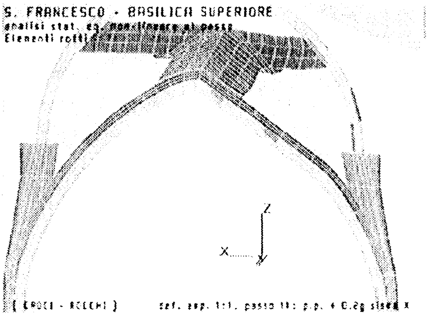

Figure 9

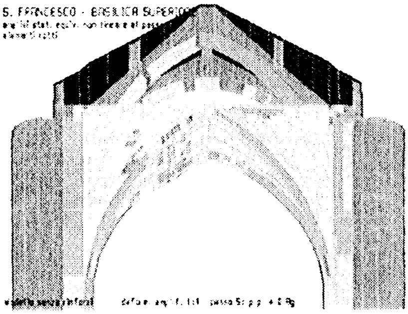

Figure 10

Particularly unidirectional aramid fibre bars were used in the intradox and unidirectional glass fibre bars in the extrados as the glass fibres have higher performances under compression stresses. By the experimental tests it has been observed how the high shear stress easily brings to the instability buckling of the lateral rib box walls if these are not stabilised by many and frequent transversal reinforcements or by an enough stiff rib core.

A solution with a stiff nucleus has been preferred either because more easy and speditive to be mounted, either because (in front of a certain weight increment) guarantees a higher resistance and energy dissipation.

According to the Mörsh theory used to calculate reinforced concrete beams under shear action, the outward layers of aramid fibre tissue act as on tension stirrups while the nucleus works as a succession of compressed inclined trusses.

Several different materials have been tested for the nucleus like rigid foams, different types of wood, compositions of wood plus foam and finally it has been chosen the mohogany nautical plywood used stratifying several layers of plywood each on the other (fig. 15). The use of plywood, instead of layers of simple wood, reduces strongly the transversal core expansion under the load action. Mohogany wood, very homogeneous, can guarantee a high resistance and stiffness to compression while the nautical treatment, realised according to the strictest nautical standards, results in an imputrescible wood of higher durability. To glue in sito flexible wooden layers allows to easily follow the vault forms and permanent deformations.

Pull tests of composite elements from masonry samples show a high adhesive power both for the direct gluing and through the aramid fibre pivots (Fig. 16). These results allowed to reduce the base rib width.

It was also tested the shear and bending bearing capacity of the timber nucleus alone which results 4÷5 times lower than those of the complete composite beam solution. On the other hand a 220x100mm section composite beam (18kg/m of weight) showed around the same yeld limit resistance which we can obtain by an IPE400 steel beam or by a HEM220 steel beam.

The experimental tests have also shown a considerable conservation of bearing capacity also after having passed the yelding point, with a good ductile behaviour even after several loading and unloading cycles.

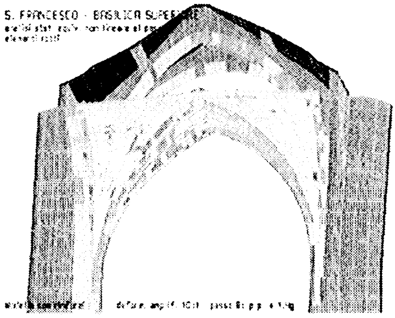

Figure 11

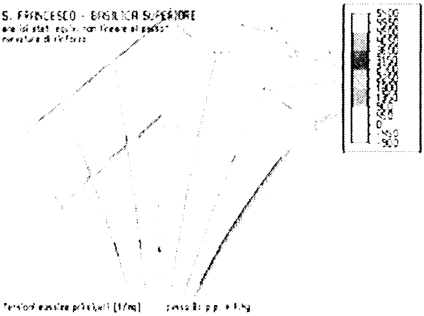

Figure 12

The final results have been positive showing excellent resistance and good bond between the ribs and masonry samples at the natural scale of the vaults (Fig. 17) and also between simple composite strips and large masonry samples.

5 The reinforcement design

A couple of ribs, of 22x10cm section each one, has been disposed in correspondence to the transversal arches (figs. 13 and 18), those more sensitive and weak in case of seismic actions because of their reduced curvature. A single rib of larger section 30x12cm has been placed just above each diagonal arch. One or two smaller ribs (section 20x5cm) are on each web, with a radial disposition from the pillars. Other joining ribs are on the vault crown (section 18x12cm) and also parallel to the crown but at a middle height of the vaults (section 12x5cm).

The ribs are directly realised in situ as the result of the stratification of various materials.

On the vault extrados surface, after an adequate cleanliness, a first four-directional (0°, ±45°, 90° angle ply) tissue of aramid fibre of 230gr/m^2 is glued using epoxy resin. Over this first layer, are glued the pultruded flat aramid fibre bars which are covered by a second tissue of aramid fibres equal to the first one. Follows the gluing and stratification of several mohogany plywood layers to realise the rib core. The wooden core is then covered by a third four-directional aramid fibre tissue of 360gr/m^2, heavier than the previous ones as it is expected to be part of the stratification of the rib lateral walls. At this point, the gluing of the pultruded flat glass fibre bars takes place. All is finally covered by another four-directional aramid fibre tissue of 360gr/m^2.

Near the springing and near the vault crown, were these ribs may suffer severe values of shear action, each rib is reinforced by gluing additional four-directional aramid fibre tissue (360gr/m^2) layers.

Each stiffening rib has been calculated as a beam placed upon the vault structural filling extrados and the vault crown. In fact, during a seismic event, the possible relative displacements of the vault crown respect to its springing doesn't allow us to take into account an arch effect for the rib itself. Moreover for great deformations it is not possible to take into account the arch effect for the vaults and the masonry arches also.

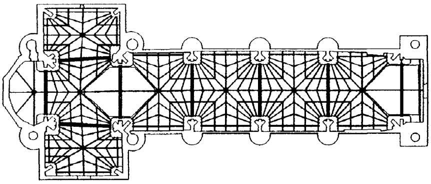

Figure 13: The ribs' net.

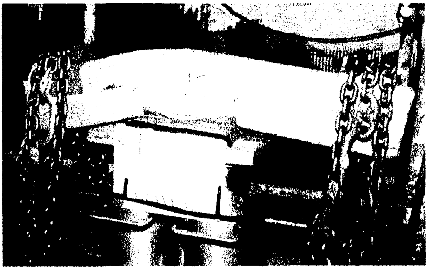

Figure 14: Rib-beam shear-bending test.

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Thus, in limit conditions, it is necessary to leave all the seismic actions on the stiffening ribs, without any collaboration from the masonry vault structures.

Designing and sizing the stiffening ribs it has been supposed to neglect the contribution of the wooden core in the bending resistance, leaving all the bending stresses as a matter of the pultruded bars. In this way, taking for instance the case of the main transversal ribs (those just above the transversal vault arch), having each one to support a moment M=6t·m and being about 22cm high, 4 aramid fibre bars (section 40x7mm) at the intradox and 6 glass fibre bars (section 40x9mm) at the extrados are necessary.

About the shear resistance it has been taken into account the wooden core and the outward stratifications in four-directional aramid tissue of 360gr/m^2. Assuming the similitude with the Mörsh theory, taking always as an example the case of the transversal ribs, each one of them has to support a maximum shear of about 5t near the springing and near the vault crown. In these positions 4 layers of aramid tissue (360gr/m^2) are necessary in the outward stratification, taking into account only the fibres on tension, while for having compatible stresses in the inclined compressed trusses, the wooden core must be 10cm wide.

In each of these separate calculations, the safety coefficients taken into account for each material, respect its yelding or rupture stress value, are in the range of 2.5÷3.5. The experimental tests have, however, clearly shown a comprehensive global safety coefficient in the range of 2÷3 between the expexted load and the yelding point load of composite beam-ribs. The same results is reached through the numerical step-by-step analysis of the vaults, described in Section 2, with a clear improvement in the vaults' behaviour. Moreover the vaults result not very deformed up to 1.4g of horizontal action, also if very partialized, because they are sustained by the ribs net.

The matemathical model also shows that large deformations of the vaults may happen only with more then 1.6g of horizontal acceleration. However, those deformations, also if they are not important for the reinforced structure of the vaults, may be dangerous for the frescos. Thus the net of ribs applied on the vault extradox will be sustained (in a certain number of points) by tie bars connected to the roof; similarly to that realized on the urgent intervention works phase.

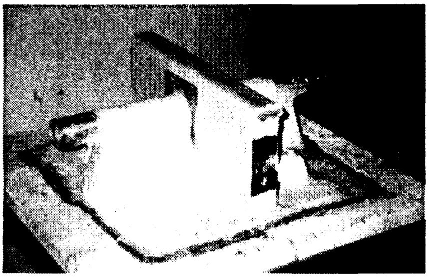

Figure 15: Sample of rib net's node.

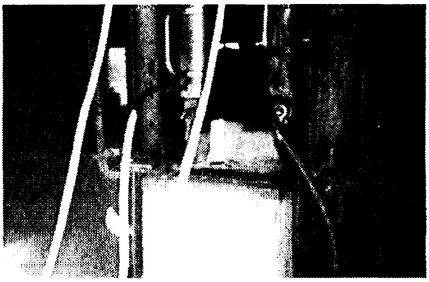

Figure 16: Pulling test.

6 Conclusions

The operations carried out, firstly to save and then to consolidate and restore the Basilica of St. Francis of Assisi, have all followed the same philosophy: to place the most up to date techniques and technologies at the service of culture in order to respect the historic value of the ancient building and to obtain adequate safety levels, changing as less as possible the original structural conception.

In particular the use of aramid fibres in composite beams, never applied before in the restoration field, have expressly been studied in this occasion, offering new interesting possibilities for the safeguard of the architectural heritage. Figure 19 shows the vaults behaviour improvement.

7

The general project of restoration has been prepared by prof. G. Croci and prof. P. Rocchi with the collaboration of ing. G. Carluccio, under the supervision of Dr. A. Paolucci, artistic co-ordinator of the Ministry of Cultural Heritage, Arch. C. Centroni, Superintendent of Umbria, and Dr. G. Basile, of the Italian Institute of Restoration. The tests and the realisation of the ribs have been carried out by SACEN s.p.a. with the collaboration of ing. A. Balsamo. We have also to thank arch. A. Herzalla and arch. M.A. Ricciardi for their collaboration in the realisation of this paper.

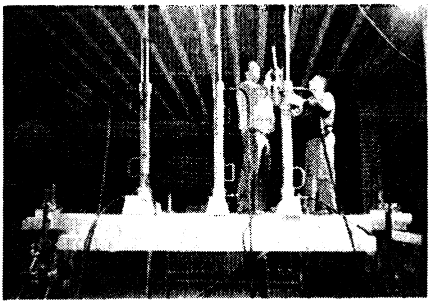

Figure 17: Rib bending test.

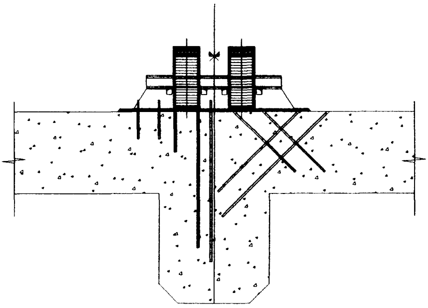

Figure 18: The transversal ribs section.

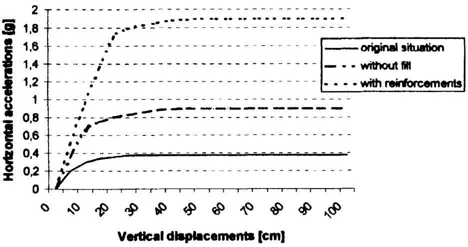

Figure 19: The improvement in the vaults behaviour.