# SHEAR STRENGTH OF DRY KEYED JOINTS AND COMPARISON WITH DIFFERENT FORMULATIONS

**MARÍA ALCALDE[*], HÉCTOR CIFUENTES[*] AND FERNANDO MEDINA[*]**

[*] Universidad de Sevilla
Escuela Técnica Superior de Ingeniería
Camino de los Descubrimientos s/n, 41092 Sevilla, Spain
e-mail: malcalde@us.es

**Key words:** Keyed Joints, Segmental Structures, Prestressed Concrete, Shear Strength

**Abstract:** The shear strength of multiple-keyed joints is a very important part of the design of prestressed segmental concrete structures. This type of structures is widely used but the formulations of different design codes deal to different values of the shear strength of joints. In this paper, it has been developed a finite element model of four different types of joints, with a number of keys varying between one and seven. The brittle cracking model was used for the material. The material model has been calibrated and validated using the P-δ curve from single edge notched beams subjected to three-point-bending test. The model has been tested comparing the predicted response with the experimental results for one and three keys. Then, it has been analysed the behaviour of joints and their dependence on the number of keys. The results have been compared with the formulation of different codes and authors. The results show that the average shear stress transferred across the dry keyed joints decreases with the number of keys but this effect is less appreciated as the compression stress acting on the joint increases. Comparing with the formulas of design codes, the ATEP formula underestimates the shear capacity of the joints, and AASHTO formula overestimates it in the case of multiple keys and low prestressing force.

## 1 INTRODUCTION

One of the most extended techniques in segmental bridge is the construction by using dry keyed joints. The speed of erection and the lack of dependency on weather conditions make this technique more suitable than the epoxied joints [1, 2].

However, the existing formulas to estimate the shear capacity of keyed dry joints from different design codes and authors lead to different values. In the literature, several experimental studies about the behaviour of keyed joints [2-5] as well as various numerical models are available [5-9]. Because the configurations of both experimental studies and numerical models are very different, a realistic comparative analysis is difficult to be done.

Regarding the Spanish design code, there is a formula recommended by ATEP [10]. This formula depends on the total area of the joint surface, without distinguishing the strength contribution of the keys:

$$V_u = A_j\left(1.14\,\sigma_n + 0.0564\sqrt{f_{cd}}\right) \tag{1}$$

Where:

$V_u$ = Ultimate shear capacity of keyed dry joint (N)

$A_j$ = Total area of the joint surface (mm$^2$)

$f_{cd}$ = Design value of concrete compressive strength (MPa)

$\sigma_n$ = Compressive stress in the joint (MPa)

In the American design codes, the formula proposed by AASHTO [11] separates the shear strength that is transmitted by the keys from the strength provided by the smooth surfaces in contact:

$$V_n = A_k \sqrt{f_{ck}}\,(0.2048\,\sigma_n + 0.9961) + \\ + 0.6\,A_{sm}\sigma_n \tag{2}$$

Where:

$V_n$ = Nominal shear capacity of keyed dry joint (N)

$A_k$ = Area of the base of all keys in the joint plane ($mm^2$)

$A_{sm}$ = Area of contact between smooth surfaces in the joint plane ($mm^2$)

$f_{ck}$ = Characteristic concrete compressive strength (MPa) ($f_c$' in [11])

Turmo 2006 [1] reviewed the different formulations available to evaluate the joint shear capacity and the experimental data published in the literature. Comparing this data with the estimated shear capacity by the ATEP [10] and the AASHTO formulas [11], Turmo proposed a new formula to be included in the Eurocode. This formula was based on the AASHTO formula, which showed the best agreement with the experimental results [1] and is given by:

$$V_n = A_k \frac{\sqrt[3]{f_{ck}^{\,2}}}{100}\,(7\,\sigma_n + 33) + 0.6\,A_{sm}\sigma_n \\ \text{if} \quad f_{ck} \le 50MPa \tag{3}$$

Rombach [12] analysed the behaviour of multiple-keyed joints by a finite element model and proposed the following formula:

$$V_n = 0.14\,A_k\,f_{ck} + 0.65\,A_j\sigma_n \tag{4}$$

Zhou [4] performed a series of experimental tests and analysed the behaviour of dry single-keyed joints and three-keyed joints. Comparing the results with the AASHTO formula, it can be seen that the shear capacity of the multiple-keyed dry joints is overestimated, especially at low confinement conditions. This is due to the fact that the formula proposed by AASHTO was derived from experimental results of single-keyed joints. This formula does not take into account the reduced capacity in multiple-keyed joints due to sequential failure. Zhou proposed to introduce a reduction factor in the AASHTO formula when the number of keys was greater than one [4].

This work deals with a new finite element model developed to estimate the shear capacity of the joints. The joint was modelled with a number of keys variation between one and seven and different prestressing stresses. The cohesive model used for concrete allowed to visualize the crack formation and propagation until the complete loss of strength of the joint. A regression equation of the results including a factor depending on the number of keys is proposed.

## 2 NUMERICAL MODEL

The material model used for concrete was a smeared crack model [13], with a tension softening behaviour in normal direction to the crack surface. The tangential behaviour model was based on the shear retention factor where the post-cracked shear stiffness decreases as the crack opening increases. The mathematical formula used for this model is the power law proposed by Rots and Blaauwerdraad [14].

In this model, when the local displacement reaches an established limit value, the element fails and it is removed from the mesh. This failure value was set to the crack opening at which the stresses in the element have already reached a zero value. This brittle failure criterion allowed to visualize the crack paths in the models.

To validate the used material model, a numerical model of a notched beam subjected to three-point bending test (Figure 1) was developed, and the results have been compared with experimental data [15].

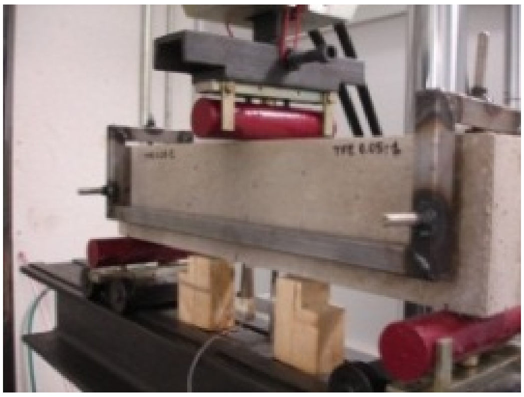

**Figure 1**: Three-point-bending test on single edge notched beams.

The numerical results are in close agreement with the experimental data (Figure 2).

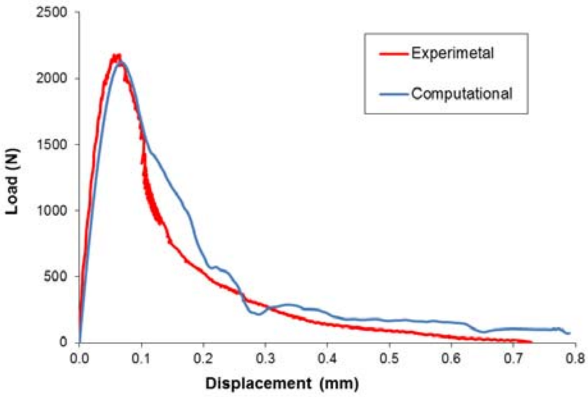


**Figure 2**: Comparison of experimental P-δ curve and numerical results of the three-point-bending test on single edge notched beams.

It has been developed a model of a joint with one and three keys to reproduce the shear joint test carried out by Zhou [4]. The model consists of two independent parts in contact as shown in Figure 3. The prestressing force is modelled as two external loads that compress the joint. The applied boundary conditions do not allow the vertical displacement of the bottom surface or the horizontal displacement of its central point.

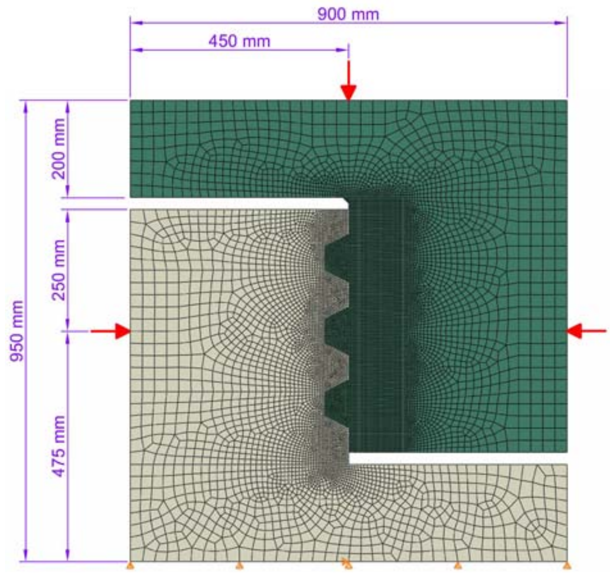


**Figure 3**: Finite element model of shear test on three-keyed joint.

The vertical load has been applied in the central point of the top surface, modelled as an imposed displacement rising from zero to the displacement that causes the failure of the joint.

To model the contact in the surface normal direction, the Hard contact model has been used. In this model, the surfaces were able to separate and to come in contact again. The model does not allow the penetration of one surface into the other one and there is no transfer of tensile stress across the interface. The tangential behaviour is based on the Coulomb friction model with a friction coefficient between concrete surfaces of $\mu$ = 0.72. This value was obtained experimentally by Zhou [4].

The material properties considered in the model are: compressive strength 50 MPa, direct tensile strength 4.5 MPa, modulus of elasticity 34.4 GPa and fracture energy 56 N/m. The stress-strain relationship for concrete in compression is based on the Sargin's curve, which has been used in Eurocode 2. To define the tensile behaviour, a linear softening branch has been used.

Tables 1, 2 and 3 show the comparison of the joint capacity numerically and experimentally obtained.

**Table 1**: Comparison of numerical and experimental results of shear capacity of single-keyed joint.

| | *Numerical model* | | | *Experimental test* | | |
|---|---|---|---|---|---|---|
| $\sigma_n$ (MPa) | 1 | 2 | 3 | 1 | 2 | 3 |
| $V_n$ (kN) | 221 | 333 | 373 | 211 | 335 | 360 |

**Table 2**: Comparison of numerical and experimental results of shear capacity of three-keyed joint.

| | *Numerical model* | | | |
|---|---|---|---|---|
| $\sigma_n$ (MPa) | 0.5 | 1 | 1.5 | 2 |
| $V_n$ (kN) | 425 | 573 | 691 | 793 |
| | *Experimental test* | | | |
| $\sigma_n$ (MPa) | 0.5 | 1 | 1.5 | 2 |
| $V_n$ (kN) | 392 | 471 | 661 | 740 |

**Table 3**: Error of the numerical results of shear capacity with regard to experimental results.

| | *Single-keyed joint* | | | |
|---|---|---|---|---|
| $\sigma_n$ (MPa) | 1 | 2 | 3 | |
| Error | 4.7 | 0.6 | 3.6 | |
| | *Three-keyed joint* | | | |
| $\sigma_n$ (MPa) | 0.5 | 1 | 1.5 | 2 |
| Error | 8.4 | 21.7 | 4.5 | 7.2 |

In most cases, the error obtained was less than 9 %. Only in the case of three-keyed joint and a prestressing stress of 1 MPa, the error is 21.7 %. It should be noted that the characteristic concrete compressive strength experimentally obtained for the cases shown in Tables 1 and 2 was 53.4 MPa with a variation coefficient of 21 %. This significantly affects to the shear capacity.

After the model validation, a new configuration corresponding to the joint with five and seven keys (Figure 4) and with a prestressing stress of 1, 2 and 3 MPa was analysed. The model of the joint with five and seven keys needs experimental verification that the authors are planning to make in further investigation.

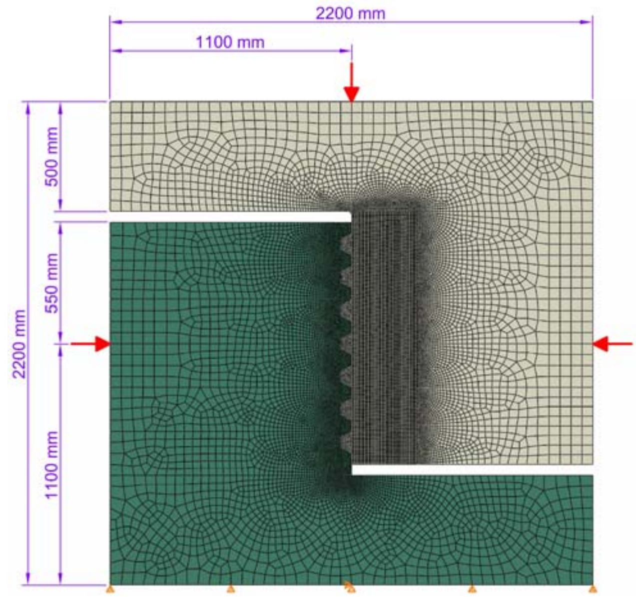


**Figure 4**: Finite element model of shear test on seven-keyed joint.

## 3 RESULTS

From the numerical results of the shear capacity for different values of the prestressing stress and the number of keys, a regression adjustment was proposed, which drive to an estimation of the shear capacity of dry keyed joints (5). This formula is only valid for concrete compressive strength of 50 MPa and a prestressing stress up to 3 MPa:

$$V_n = 7.118\,A_k\left(1 - 0.064\,N_k\right) + + 2.436\,A_{sm}\sigma_n\left(1 + 0.127\,N_k\right) \tag{5}$$

Where:
$N_k$ = Number of keys in the joint.

Comparing the numerical values with those obtained from the formula (5) it can be seen that the formula reproduces the numerical results with a maximum error of 8.7 %. This formula describes the joint behaviour depending on the number of keys and the prestressing stress.

To compare the estimations given by the ATEP formula (1), which are ultimate values, with the nominal values from the AASHTO formula (2), the ATEP estimations must be reduced by using the strength reduction factor from the AASHTO (0.75). Figures 5, 6, 7 and 8 show the comparison of the shear capacity obtained from the estimation formula (5) with the formulas proposed by ATEP (1), AASHTO (2), Turmo (3) and Rombach (4), for 1, 3, 5 and 7 keys and depending on the prestressing stress.

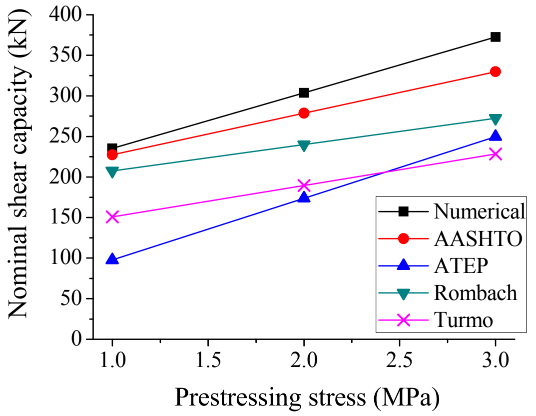


**Figure 5**: Nominal shear capacity versus prestressing stress for single-keyed joint.

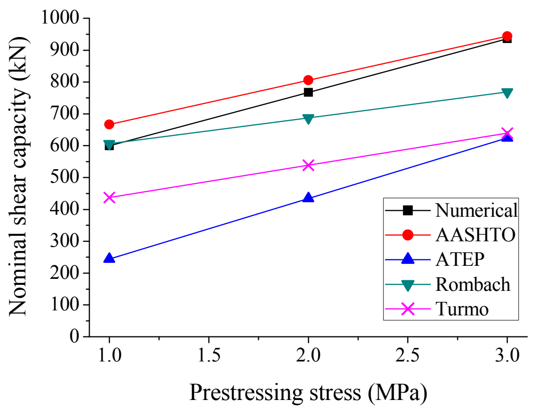


**Figure 6**: Nominal shear capacity versus prestressing stress for three-keyed joint.

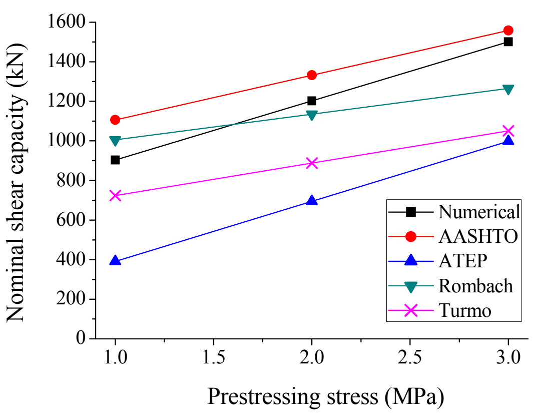


**Figure 7**: Nominal shear capacity versus prestressing stress for five-keyed joint.

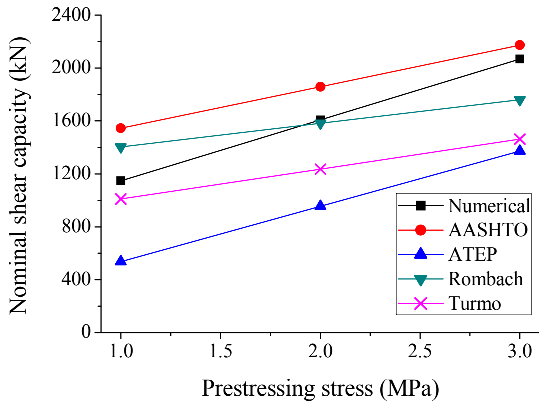


**Figure 8**: Nominal shear capacity versus prestressing stress for seven-keyed joint.

The results from the AASHTO formula (2) overestimate the shear capacity of multiple-keyed dry joints, such as Zhou demonstrated experimentally [4]. On the other hand, the estimations from the ATEP formula (1) remain far on the conservative side [1]. The values from the formula proposed by Rombach (4) are closer to the numerical results, but for 5 and 7 keys there are some values bigger than the numerical ones. The formula proposed by Turmo (3) gives values that are more conservative for a small number of keys.

The results showed a dependency on the number of keys. For prestressing stress lower than 3 MPa, it should be necessary to introduce a coefficient to consider this dependency [4], as it occur in the regression equation proposed in this paper (5).

The average shear stress transferred across the joint obtained from the estimation formula (5) decreases as the number of keys increases (Figure 9). However, this effect declines gradually as the compressive stress in the joint increases. In the case of 3 MPa, the average shear stress becomes independent of the number of keys. This is due to the fact that a high compressive stress introduces a more plastic behaviour in the joint [16] and therefore all the keys are able to develop their full capacity. This can also be seen in the numerical model where the keys fail almost at the same time and not sequentially.

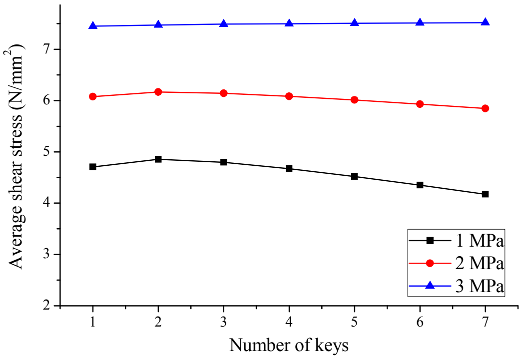


**Figure 9**: Average shear stress transferred across the joint versus number of keys for 1, 2 and 3 MPa of prestressing stress.

## 4 CONCLUSIONS

The average shear stress transferred across the dry keyed joints decreases as the number of keys increases, due to the sequentially failure of the keys. However, the current available formulas do not include any factor that takes into account this dependency.

A numerical model of a joint with 1, 3, 5 and 7 keys was developed, and a regression equation of the obtained results was proposed. This equation estimates the shear capacity depending on the number of keys and the prestressing stress (until 3 MPa) for a characteristic concrete compressive strength of 50 MPa.

The average shear stress transferred across the joint decreases as the number of keys increases, but this effect declines with higher prestressing stress in the joint, becoming negligible for 3 MPa. Because of this, if the prestressing stress is higher than 3 MPa, it is not necessary to include any correction considering the dependency of the shear strength on the number of keys.

## 5 ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support provided for this research by the Spanish Ministry of Science and Technology under the project BIA2010-21399-C02-02.

## REFERENCES

[1] Turmo, J., Ramos, G. and Aparicio, A. C., 2006. Shear strength of match cast dry joints of precast concrete segmental bridges: Proposal for Eurocode 2. *Materiales de Construcción*, **56**, 282, pp. 45-52.

[2] Buyukozturk, O., Bakhoum, M. M. and Beattie, S. M., 1989. Shear behavior of joints in precast concrete segmental bridges. *Journal of structural engineering New York, N.Y.*, **116**, 12, pp. 3380-3401.

[3] Turmo, J., Ramos, G. and Aparicio, A. C., 2006. Shear strength of dry joints of concrete panels with and without steel fibres. Application to precast segmental bridges. *Engineering Structures*, **28**, 1, pp. 23-33.

[4] Zhou, X., Mickleborough, N. and Li, Z., 2005. Shear strength of joints in precast concrete segmental bridges. *ACI Structural Journal*, **102**, 1, pp. 3-11.

[5] Issa, M. A. and Abdalla, H. A., 2007. Structural behavior of single key joints in precast concrete segmental bridges. *Journal of Bridge Engineering*, **12**, 3, pp. 315-324.

[6] Kaneko, Y., Connor, J. J., Triantafillou, T. C. and Leung, C. K., 1993. Fracture mechanics approach for failure of concrete shear key. I: theory. *Journal of Engineering Mechanics*, **119**, 4, pp. 681-700.

[7] Kaneko, Y., Connor, J. J., Triantafillou, T. C. and Leung, C. K., 1993. Fracture mechanics approach for failure of concrete shear key. II: verification. *Journal of Engineering Mechanics*, **119**, 4, pp. 701-719.

[8] Turmo, J., Ramos, G. and Aparicio, A. C., 2006. FEM modelling of unbonded post-tensioned segmental beams with dry joints. *Engineering Structures*, **28**, 13, pp. 1852-1863.

[9] Rombach, G. and Specker, A., 2000.

Finite Element Analysis of Externally Prestressed Segmental Bridges. In *Proceedings of the Proceedings of the Fourteenth Engineering Mechanics Conference (EM 2000)*, 2000, Austin.

[10] ATEP, 1996. *Proyecto y construcción de puentes y estructuras con pretensado exterior. H.P.10-96.* Colegio de Ingenieros de Caminos, Canales y Puertos, Madrid.

[11] AASHTO, 1999. *Guide Specifications for Design and Construction of Segmental Concrete Bridges*. American Association of State Highway and Transportation Officials.

[12] Rombach, G., 2004. Dry joint behavior of hollow box girder segmental bridges. In *Proceedings of the FIP Symposium "Segmental Construction in Concrete"*, 2004, New Delhi.

[13] Bažant, Z. P. and Oh, B. H., 1983. Crack band theory for fracture of concrete. *Materials and Structures*, **16**, 3, pp. 155-177.

[14] Rots, J. G. and Blaauwendraad, J., 1989. Crack models for concrete: discrete or smeared? Fixed multi-directional or rotating?. *Heron*, **34**, 1, pp. 3-59.

[15] Cifuentes, H., Medina, F., Montero, J. R. and Piqueras, J. L., 2008. Effect of the properties of fibers on size effect of high strength polypropylene fiber reinforced concrete. In *Proceedings of the 8th International Symposium on Utilization of High-Strength and High-Performance Concrete*, 2008, Tokyo, pp. 595-602.

[16] Rombach, G., Specker, A., Zhou, X., Mickleborough, N., Li, Z., Turmo, J., Ramos, G. and Aparicio, Á. C., 2005. Discussion on 'Shear strength of joints in precast concrete segmental bridges'. *ACI Structural Journal*, **102**, 6, pp. 901-904.