LARGE SCALE QUASI-STATIC CYCLIC TESTS OF EXISTING BRIDGE PIERS

M. Bimschas[1] and A. Dazio[1]

[1] *Institute of Structural Engineering, ETH Zürich, Switzerland*
Email: bimschas@ibk.baug.ethz.ch, dazio@ibk.baug.ethz.ch

ABSTRACT:

Older existing bridge piers often show detailing which from today's perspective would no longer be acceptable for seismic design. Two of the most common deficiencies are low transverse reinforcement ratios and lap-splices at the pier base. The former may cause premature shear failure, thus reducing the displacement capacity of the pier, while the latter can lead to a degradation of flexural strength during inelastic cyclic loading which can cause an increase in displacement demand. As the flexural strength defines the shear demand there may also be an interaction between the lap-splice behavior and potential shear problems. In this paper the results of two large scale quasi-static cyclic tests on bridge piers are presented. Both test units had essentially the same design featuring a low transverse reinforcement ratio with the only difference being that one unit had a lap-splice at the base while the other one had continuous reinforcement. This way a potential interaction between lap-splice behavior and potential shear problems could be studied. A comparison of the experimental results with existing models to describe both failure modes is presented.

KEYWORDS: Bridge piers, large-scale tests, lap-splice, shear capacity, seismic assessment

1. INTRODUCTION

Significant progress could be achieved in the past decades concerning the development of models and guidelines for the seismic design of new structures. In particular, modern capacity design principles ensure that undesirable failure modes are prevented and the structure behaves in a reliable and well predictable manner. Unfortunately, older existing bridge structures often do not comply with these principles concerning design and detailing. As a consequence, the response models developed for modern structures may not always be directly applicable for the assessment of existing bridges. Typical detailing deficiencies found in existing bridge piers include insufficient transverse reinforcement and lap-splices in the potential plastic regions.

From damage occurred during previous earthquakes and experimental investigations it is well known that shear failure as well as lap-splice failure in the plastic zone of existing bridge piers are common failure modes under seismic loading. Cyclic inelastic shear strength has already been the subject of various experimental studies on bridge piers as well as building columns which resulted in several models for the shear strength, explicitly taking into account the inelastic deformation demand in terms of ductility (e.g. Aschheim and Moehle, 1992; Kowalsky and Priestley, 2000; Sezen and Moehle, 2004).

Likewise, cyclic failure of lap-splices in plastic regions has been studied experimentally and a model to describe its influence on the cyclic strength degradation of the member has been proposed in Priestley *et al.* (1996). It should be mentioned that many experimental studies (e.g. Chai *et al.*, 1991; Lynn *et al.*, 1996; Melek and Wallace, 2004) have been conducted with short lap-splices having lengths of 20 d_b which were common in the U.S. in previous decades as pure compression splices. These lap-splices would not comply with common code provisions for tension splices even under static loads. However, in Switzerland for example, even in the past compression as well as tension lap-splices have been designed with a length of at least 40 d_b which should warrant to some extent a better performance than that of the short lap-splices mentioned above.

Another important issue is a potential interaction between lap-splice and shear behavior. While Chai *et al.* (1991) and Melek and Wallace (2004) did not observe any shear problems in their test units with lap-splices, Lynn *et al.* (1996) classified all of their column failures as shear failures. According to their observations, in some cases the shear failure initiated from the lap-splice failure.

At the same time, based on theoretical considerations it might also be possible that the cyclic strength degradation which is typical for lap-splice failure could prevent shear problems. If a shear failure occurs in the inelastic deformation range it is the result of several cycles at large displacements combined with high shear forces. However, the degradation of the lap-splices could reduce the shear demand at increasing deformations and thus also reduce the damage outside the splice region which would otherwise lead to the shear failure. This is of importance for two reasons: (1) Despite of the decrease in horizontal force capacity, the displacement capacity may be increased, and (2) a loss of axial load capacity, which is often the result of a shear failure, might be prevented.

Table 1 Characteristic data of test units

		VK1	VK2
Axial load	N [kN]	1370	1370
Concrete compressive cylinder strength	f_c [MPa]	35	39
Yield strength of longitudinal reinforcement	$f_{sy,l}$ [MPa]	515	515
Ultimate strength of longitudinal reinforcement	$f_{su,l}$ [MPa]	630	630
Yield strength of transverse reinforcement	$f_{sy,t}$ [MPa]	500	500
Axial load ratio	n [-]	0.075	0.067
Longitudinal reinforcement ratio	ρ_l [%]	0.82	0.82
Transverse reinforcement ratio	ρ_t [%]	0.08	0.08
Aspect ratio	H/l_w	2.2	2.2
Lap-splice length	l_s [mm]	-	600

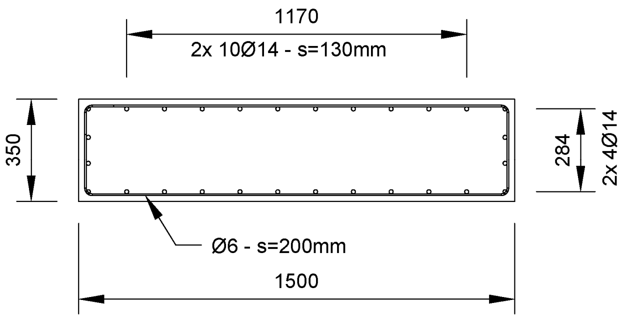

Figure 1.a Cross-section VK1, VK2
(all dimensions in [mm])

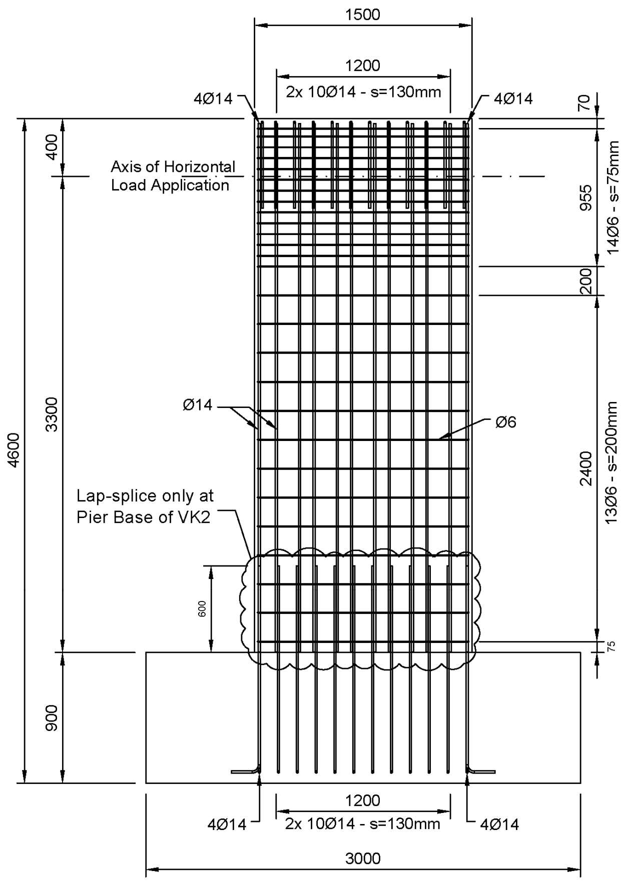

Figure 1.b Elevation VK2
(all dimensions in [mm])

In the following chapters the experimental results of two of the quasi-static cyclic tests conducted within an ongoing research project at ETH Zürich are presented. The goal of these two tests was to study the cyclic inelastic shear strength of existing wall type bridge piers with and without lap-splices at the base.

2. DESCRIPTION OF TEST SERIES

The test units were designed to represent real existing Swiss bridge piers from the 1960s with detailing incorporating seismic deficiencies which are typical for structures of that time. All units have the same concrete dimensions, same steel properties and similar concrete properties. The scale of the models with respect to the hypothetical prototypes is 50%. The layout of the transverse reinforcement was the same for all units tested. Also the longitudinal reinforcement layout, in terms of diameter und number of bars, was the same for test units VK1 and VK2, the only difference being that VK1 had continuous longitudinal reinforcement from the foundation up to the pier top, while VK2 was provided with a lap-splice at the pier base. The splice length was

600 mm which equals 43 d_b and relates to the minimum splice length of 40 d_b required by current and previous Swiss code provisions. The details of the design data for test units VK1 and VK2 can be found in Table 1 and Figure 1.

Particularly noteworthy is the very low transverse reinforcement ratio of 0.08% which provides rather little contribution to the total shear strength. The hoops are closed by simple 90° hooks and no additional confinement reinforcement in the end zones of the cross-section is provided. Aside from some confinement effect provided by the foundation block at the base, the concrete can thus be considered mainly unconfined.

The loading history consisted of loadsteps with increasing displacement amplitude, whereas two cycles were conducted at every step. Between some of the large cycles in the inelastic range, two small cycles were added to study the small cycle response after previous large cycles. It is not believed that these small cycles have influenced the overall result in terms of displacement capacity or damage behavior.

3. TEST RESULTS

Although the only difference between the two test units was the lap-splice at the base (and a small difference in concrete compression strength) their behavior during the experiment differed significantly.

In Figure 2, both test units can be seen in their final state at the end of the tests and Figure 3 shows the hysteretic responses resulting from the applied loading histories. Generally, the behavior of VK2 can be divided into a phase before the onset of lap-splice failure and one after it. Before the degradation starts, both units behave in a similar way. However, the response of test unit VK2 featuring the splice is a little stiffer than that of VK1. This is due to the fact that before the damage starts, the lap-splice is actually longer than required to transfer the full force from the starter bars to the pier bars, leading to a concentration of the force transfer to both end-zones of the lap-splice length. Between those end-zones, basically both bars are fully active thus doubling the total amount of longitudinal reinforcement along the inner part of the splice length. In the elastic range this increased longitudinal reinforcement close to the base leads to a certain stiffening of VK2 compared to the case of continuous reinforcement as in VK1. In the inelastic range and before the onset of degradation, the doubled reinforcement close to the base leads to a reduction of the plastic zone by preventing the plastic steel strains to spread upwards into the pier. Due to the thus reduced plastic hinge length higher steel strains are required for the same top displacements compared to VK1.

At a drift ratio of approximately 1%, which relates to a displacement ductility of $\mu_\Delta \approx 4$ (based on an experimentally determined nominal yield displacement for VK2 of $\Delta_y = 8.2$ mm), the first lap-splices started to fail leading to the onset of flexural strength degradation of VK2. While the first excursion to the south still showed a stable behavior with even increasing moment capacity, during the following loading to the north a clear degradation of flexural strength was observed. From then on the strength degraded more and more in every further cycle as can be seen in Figure 3.b. In parallel, the failure of the lap-splices progressed inwards to the center region of the cross-section until at the end of the experiment almost all splices had completely failed and the remaining flexural resistance resulted from eccentricity of the axial load only. At the same time, damage of the compression zones progressed inwards as well, leaving only a relatively small concrete core in the center of the cross-section to support the pier (see Figure 2.b). Nevertheless, even this limited and also heavily damaged core was still able to support the full axial load of 1370 kN up to the end of the experiment. The test of VK2 was finally stopped after two full cycles at a drift ratio of δ = 3.2% which relates to a displacement ductility of $\mu_\Delta \approx 13$. A real failure did not occur for VK2. However, at the end the horizontal resistance had dropped to approximately 15% of the maximum force capacity measured during the experiment.

In contrast to this, test unit VK1 featuring the continuous longitudinal reinforcement did not show any significant strength degradation during most part of the test but rather developed stable hysteretic loops in the inelastic range as well. Up to a drift ratio of δ = 1.6% which corresponds to a displacement ductility of $\mu_\Delta \approx 5.4$ ($\Delta_y = 9.7$ mm, for VK1), the behavior was mainly flexure dominated. A complete crack pattern, including inclined shear cracks, had developed during early stages. However, up to δ = 1.6%, these shear cracks did not open up more than approximately 1 to 1.5 mm, while at the same time the flexural cracks had already reached widths of 3 to 5 mm. During large part of the test no significant shear distress was visible and the damage was rather concentrated to buckling of longitudinal reinforcement and loss of concrete compression zone due to flexural deformations.

Figure 2 Test unit VK1 (a) after failure and test unit VK2 (b) after end of test

This behavior changed quite rapidly at a drift ratio of δ = 1.9 % (μ_Δ = 6.5) when during the north excursion of the first cycle a significant strength degradation started to develop. This was accompanied by significant growth of shear crack widths indicating an imminent shear failure. This change in behavior was initiated by the loss of flexural compression zone which at this stage had advanced thus far that the inclined compression strut which carried the shear force into the foundation block started to loose its support. The pier still survived the second cycle at a drift ratio of δ = 1.9% with significantly reduced horizontal force capacity, but finally failed during the attempt to proceed to the next loadstep before reaching a drift ratio of δ = 1.9% again. The failure occurred by a sudden opening of a shear crack with all crossing hoops fracturing and the upper part of the pier sliding down the diagonal failure crack. At the same time, both the shear and the axial load capacity were lost immediately. In case of a real bridge pier, the inability to carry the gravity load from the superstructure anymore would have resulted in a collapse of at least the two neighboring bridge spans.

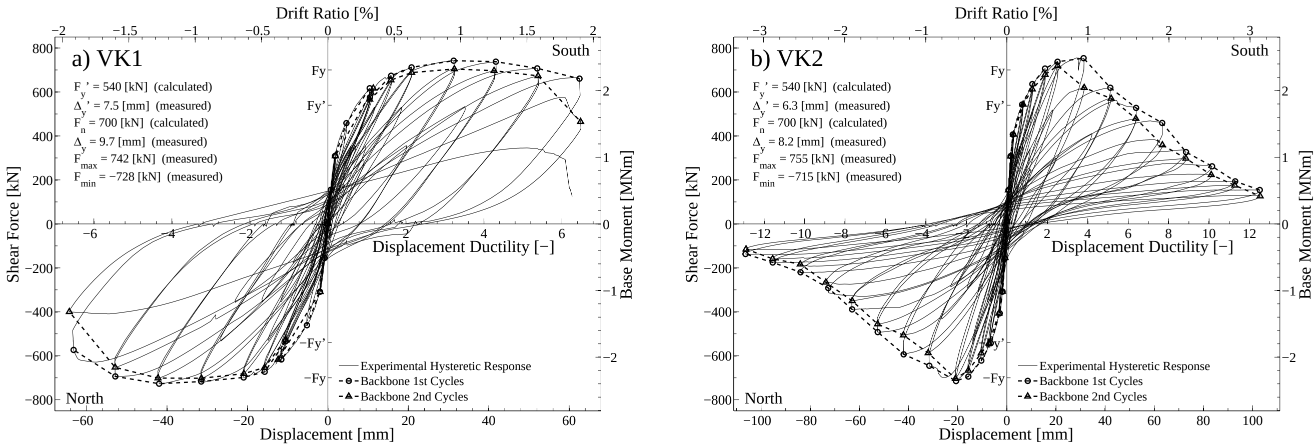

Figure 3 Measured hysteretic response of VK1 (a) and VK2 (b)

4. INTERPRETATION OF TEST RESULTS

Comparing the results of the two test units VK1 and VK2, it can be seen that the consideration of chapter 1 concerning a potential prevention of shear failure by lap-splice degradation can indeed occur in certain cases. However, considering the experimental results by Lynn *et al.* (1996), it should be kept in mind that this effect cannot be generalized as Lynn observed in his experiments shear failure occurring even after lap-splice failure. In an individual case the occurrence of this phenomenon will depend on the relationship between shear demand and capacity as well as on the point of lap-splice failure and the rate of its degradation. As a consequence, longitudinal and transverse reinforcement ratios as well as lap-splice length will (among others) strongly influence the interaction between shear and lap-splice behavior.

Even in cases where a prevention of shear failure by lap-splice degradation is deemed possible, it may not always be clear which behavior should actually be considered more critical and it may depend on the characteristics of the whole bridge structure in each individual case. On one hand, the prevention of a potential shear failure is certainly beneficial since it is likely to be accompanied by loss of axial load capacity which would lead to a total collapse of at least the neighboring bridge spans. On the other hand, up to the point of shear failure, a pier with continuous reinforcement might show a more stable hysteretic behavior with less strength degradation, while a pier with failing lap-splices may develop significant degradation of horizontal force capacity already at rather low deformation levels, thus leading to an increase of displacement demand. Care should also be taken in relying on the axial load capacity of piers with lap-splices at high deformation levels. Although VK2 was able to carry its full axial load up to the end of the experiment, it should be noted that a) the axial load ratio of 7.5% in this test was not very high, b) towards the end of the test the remaining concrete core carrying the axial load was only consisting of loose concrete debris with rather low reliability (especially under real dynamic loading), and c) the tests were conducted with a transverse support at the pier top ensuring that only unidirectional deformations were applied to the test unit. Considering the severe damage to the pier base of VK2 at the end of the test, it may not be warranted that under real dynamic, bidirectional loading the axial load could still have been carried at these stages as well.

In an individual assessment case, the evaluation of piers with degrading lap-splices may also depend on the bridge structure as a whole. If for example other bridge piers or the abutments are capable of providing horizontal support while the degrading pier is progressively developing a hinge at its base, the overall behavior may still be acceptable as long as it will not fail under gravity loads. On the other hand, the uncertainties related to the response prediction should be taken into account, especially if an interaction between shear and lap-splice failure cannot be ruled out. Therefore, in certain cases it may be indicated to analyze the structure with various assumptions considering either shear or lap-splice failure – or, as observed in Lynn's experiments, a combination of both failure modes. In the following chapter, the results of VK1 and VK2 are compared to existing models for cyclic shear failure as well as lap-splice degradation. These models may also serve as preliminary means for assessment cases.

5. COMPARISON WITH EXISTING FAILURE MODELS

In Figure 4 the backbones of the measured force-displacement responses for VK1 and VK2 are compared to the ductility dependent shear capacity models by Kowalsky and Priestley (2000) as well as Sezen and Moehle (2004). Both models are applied using two different nominal yield displacements Δ_y to define the displacement ductilities corresponding to a given top displacement. In the first case, the nominal yield displacements were calculated based on numerical moment curvature analysis and the procedure outlined in Priestley *et al.* (1996). In the second case, measured displacements from the experiments were used to define the displacement Δ'_y at first yield. In this case, the corresponding nominal yield displacement was calculated as $\Delta_y = \Delta'_y \cdot (M_n/M'_y)$, where M'_y and M_n are the first yield and nominal moment capacities respectively, as obtained from moment-curvature analysis (Priestley *et al.*, 1996).

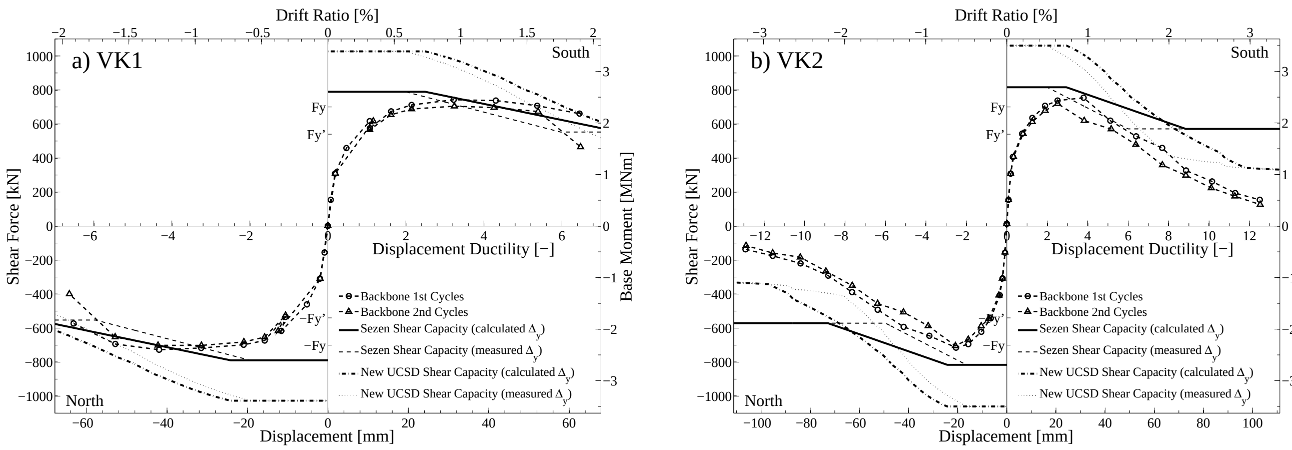

Figure 4 Comparison of the response of VK1 (a) and VK2 (b) with shear capacity models

In Figure 4 it can be seen that the model by Sezen and Moehle (2004) predicts at low to medium ductility demands lower shear capacities than the model by Kowalsky and Priestley (2000). In this context it should be noted that in both models the concrete part of the shear capacity depends on the aspect ratio of the member. While Sezen and Moehle (2004) use the distance d between the compression edge of the cross-section and the centroid of the longitudinal tension reinforcement to calculate the aspect ratio a/d, Kowalsky and Priestley (2000) use the total section depth h for the same purpose. For beams and building columns with concentrated reinforcement layers the difference may not be large. However, for wall type cross-sections with distributed reinforcement along the edges, the difference between d and h may become significant with d even depending on the load level. In the case of Figure 5 the Sezen model was applied using the approximation of $d = 0.8h$ as suggested by ACI 318 (2005) for circular cross-sections. If in turn the total section depth was used for the calculation of the aspect ratio the Sezen model would yield higher values by 25% for the concrete part, thus resulting in similar values for low to medium ductility demands as the model by Kowalsky and Priestley (2000). On the other hand, in this case the difference between the two models at high ductility demands would be even larger.

It can be seen from Figure 4 that all applications of the two models intersect the measured response curve of VK1 and would thus suggest a shear failure in the inelastic range. Depending on the model and the type of ductility definition, they only differ in the predicted displacement at failure, with the model by Kowalsky and Priestley (2000) being somewhat closer to the observed failure displacement in the experiment. For VK2, on the other hand, three of the four model applications do not intersect the measured response curves. Only the model by Sezen and Moehle (2004) in combination with the measured nominal yield displacement intersects the backbone during the south excursion of the 1st cycle in a rather small range tangentially. It may thus be said that in combination with the measured force-displacement curves of VK1 and VK2 both models were more or less able to predict correctly whether a shear failure occurred or not. Care should be taken though when estimating displacements at shear failure from these models as the uncertainties may be large.

To estimate the degrading behavior of a member with lap-splices in the plastic zone, Priestley *et al.* (1996) suggested a model which is based on the hypothesis that the distress of lap-splices under cyclic loading is a result of concrete compression strains higher than 0.002 in previous cycles. At this strain level the concrete typically develops micro cracks which lead to a softening behavior and also reduce its tension capacity. Since the force transfer by lap-splices depends on the tension strength of the concrete (unless significant confinement is provided) these micro cracks are considered to cause the degradation of the lap-splice.

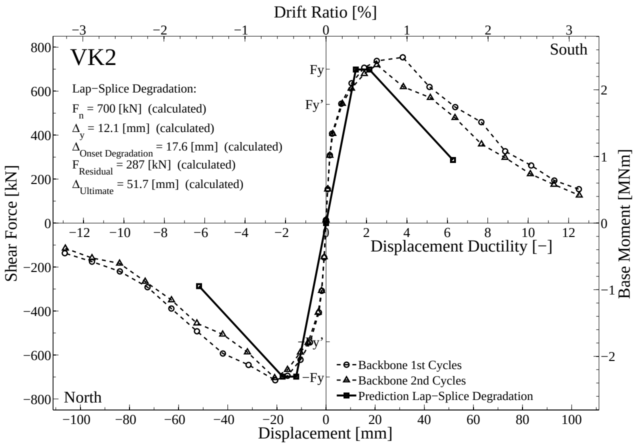

Figure 5 Comparison of VK2 response with the model for lap-splice degradation by Priestley *et al*. (1996)

The hypothesis that previous concrete compression strains are responsible for the splice degradation is supported by the behavior of VK2. Whereas during the first excursion to the south at δ = 0.95% no strength degradation was observed, the following excursion to the north at the same displacement level already showed a clear reduction of horizontal force capacity. In this north excursion, the tension zone had previously been subjected to compression strains of a level which the corresponding tension zone of the preceding south excursion had not experienced before. Only during the second cycle the tension zone of the south excursion had also been subjected to equivalent compression strains and this is when the response to the south at δ = 0.95% also showed strength degradation. It may be noted that the remaining force capacity of 619 kN in the south excursion of the second cycle is rather close to the corresponding value of 645 kN during the north excursion of the 1st cycle. Also, the force degradation between 1st and 2nd cycle at δ = 0.95% is much more pronounced for the south excursion (ΔF = 134 kN) than for the north excursion (ΔF = 58 kN) as in the latter case a larger degradation had already taken place during the 1st cycle. This behavior suggests that it may indeed be the preceding concrete compression strains that strongly influence the cyclic lap-splice degradation.
In Figure 5 the model proposed by Priestley *et al.* (1996) for the response prediction of members with lap-splices in the plastic zone is compared with the measured response of VK2. Up to the onset of strength degradation the predicted relationship consists of a bilinear approximation of the force-displacement curve based on a plastic hinge model without particular consideration of the lap-splice. The curve starts to decrease when at the base a compression strain of 0.002 is reached. It is assumed that the base curvature can be further increased by eight times the nominal yield curvature until a residual moment capacity is reached. Between the onset of degradation and the residual capacity, a linear relationship is assumed. The residual moment capacity at the base results from eccentricity of the axial force only.
For Figure 5, the displacements and forces at the mentioned characteristic strain and curvature limits have been determined by pure numerical analysis using measured material strengths for the moment curvature analysis. The model predicts an earlier onset of lap-splice failure as well as a somewhat higher rate of strength degradation. Since it was rather calibrated on the behavior of short compression lap-splices, as discussed in chapter 1, it is not surprising that the longer lap-splices provided in VK2 performed better than predicted by the model. Thus, for the flexural strength degradation this model can be considered conservative for the given case of VK2 with its longer lap-splices. However, due to this conservatism the risk of an inelastic shear failure may be underestimated as the flexural force-displacement curve also represents the shear demand.

6. CONCLUSIONS

The test results presented in this paper show that potential shear problems under seismic loading may be influenced by degradation of lap-splices in the plastic zone of bridge piers. The reduction in shear demand due to the decreasing flexural capacity can prevent a shear failure which would otherwise occur with continuous longitudinal reinforcement. This leads to a complete change of the failure mechanism having positive as well as negative consequences. Especially a prevention of premature axial load failure which often accompanies shear failures would be considered a beneficial consequence of the lap-splice degradation. On the other hand, the rather early loss of flexural capacity which can e.g. increase the displacement demand is clearly a negative consequence which might become critical, particularly if the bridge structure cannot provide horizontal support from other piers or the abutments.

Currently existing models for cyclic-inelastic shear capacity were able to generally describe the shear behavior of the tested models of wall type bridge piers. However, the uncertainty in particular with respect to the displacement at failure is significant. It may therefore be recommendable to apply them with caution as a potential axial load failure which is likely to accompany a shear failure can have disastrous consequences. It should also be mentioned that the discussed models for cyclic-inelastic shear capacity were calibrated on members with cross-sections differing from the wall-type piers investigated in this study. Further experimental verification for this latter type would therefore be advisable.

An existing model for strength degradation due to lap-splice failure seems to describe the general trend of the behavior adequately and the reasoning behind the model was supported by the test results. At the current state, this model does not yet allow for the length of the lap-splice nor for other parameters which are known to influence the strength of lap-splices, such as e.g. bond conditions, concrete tension strength or concrete cover. This may to some extent explain the quantitative differences between observed experimental results and model prediction. Further refinement of the model may overcome the currently remaining shortcomings, making it also applicable for an improved estimation of the relationship between lap-splice degradation and cyclic-inelastic shear failure.

ACKNOWLEDGEMENTS

The research project leading to this paper has been funded by the Swiss Federal Roads Authority (ASTRA) under award number AGB 2003/014 and by ETH Zurich. This financial support is gratefully acknowledged.

REFERENCES

ACI 318 (2005). Building Code Requirements for Structural Concrete. American Concrete Institute, Committee 318, Farmington Hills, Michigan, USA.

Aschheim, M. and Moehle, J.P. (1992). Shear Strength and Deformability of RC Bridge Columns Subjected to Inelastic Cyclic Displacements. UCB/EERC-92/04. Earthquake Engineering Research Center, UC Berkeley.

Chai, Y.H., Priestley, M.J.N., and Seible, F. (1991). Seismic Retrofit of Circular Bridge Columns for Enhanced Flexural Performance. *ACI Structural Journal* **88:5,** 572-584.

Kowalsky, M.J. and Priestley, M.J.N. (2000). Improved Analytical Model for Shear Strength of Circular Reinforced Concrete Columns in Seismic Regions. *ACI Structural Journal* **97:3,** 388-396.

Lynn, A.C., Moehle, J.P., Mahin, S.A. and Holmes, W.T. (1996). Seismic Evaluation of Existing Reinforced Concrete Building Columns. *Earthquake Spectra* **12:4,** 715-739.

Melek, M. and Wallace, W. (2004). Cyclic Behavior of Columns with Short Lap Splices. *ACI Structural Journal* **101:6,** 802-811.

Priestley, M.J.N., Seible, F. and Calvi, G.M. (1996). Seismic Design and Retrofit of Bridges, John Wiley & Sons, New York, USA.

Sezen, H. and Moehle, J.P. (2004). Shear Strength Model for Lightly Reinforced Concrete Columns. *Journal of Structural Engineering* **130:11,** 1692-1703.