RESPONSE OF R.C. BRIDGE PIERS TO
LARGE DEFLECTION REVERSALS

By

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ABSTRACT

A total of sixteen large-scale reinforced concrete specimens without web reinforcements were tested to study the behaviour of typical bridge piers subjected to deflection reversals large enough to cause extensive yielding of the longitudinal reinforcement. The different parameters considered included shear span-to-depth ratio, percentage of longitudinal reinforcement, frequency of the applied load, and axial compressive stresses.

Based on a parametric study of the experimental results, a nondimensional factor was introduced to describe the fundamental behaviour of bridge pier structures within the described category. The proposed characteristic factor was used to evaluate the ductility, maximum shear stress, and energy dissipation for such members. The three modes of failure observed were also classified according to the range of the proposed factor.

The effect of the load frequency indicated possible changes in the failure criteria under higher frequencies. The presence of axial compressive stresses appeared to increase the maximum shear resistance and ductility of such members and affected the modes of failure.
Keywords: Axial compression; cyclic deflection; ductility, energy dissipation; load frequency; reinforced concrete; shear failure.
INTRODUCTION

Bridge piers are commonly lightly reinforced in both longitudinal and transverse directions. Unlike beams and columns, they fall into different categories in terms of shear span-to-depth ratio, as related to the percentage of reinforcement and level of axial compressive stresses (Ozaka, 1979). Very little information is available to describe their response to large cyclic deflection reversals.

The main objective of this paper is to study the behaviour of typical bridge piers when they are subjected to deflection reversals large enough to cause extensive yielding of the longitudinal reinforcements. A total of sixteen large-scale reinforced concrete specimens without web reinforcement were tested at the University of Manitoba in a research program sponsored by the Natural Sciences and Engineering Research Council of Canada. The specimens were representative of typical bridge piers in terms of material properties, section properties, and construction details. Different parameters believed to have an influence on their behaviour were considered, such as shear span-to-depth ratio \( (a/d) \), percentage of longitudinal reinforcement \( (p) \), frequency of the applied load, and level of axial compressive stresses.

The experimental program was divided into three major series. The first series of tests was subdivided into two
subseries. The objective for subseries I-A was to examine the effect of the shear span-to-depth ratio, $a/d$, on the mode of failure and ductility. Three specimens with different $a/d$ values were tested, as given in Table 1. For these specimens, the percentage of longitudinal reinforcement was varied to maintain a constant ratio between the shear strength under monotonic load, $v_c$, and the shear stress at yielding of the longitudinal reinforcement, $v_y$. The chosen ratio ensured that the potential shear strength of the specimens was higher than the shear stress required to yield the longitudinal reinforcement. In subseries I-B, seven specimens were tested to examine the effect of the percentage of steel, $p$.

The effect of the load frequency was examined in Series II. Four specimens were tested with the different frequencies available with the testing machine used, as given in Table 1. Finally, two specimens were tested in Series III with axial compressive force, which induced a compressive stress equivalent to 0.98 MPa to simulate the actual conditions of the prototype (Ozaka, 1979).

**TEST SPECIMENS**

The relationship between a tested specimen and a bridge pier is shown in Figure 1. The dimensions and reinforcement details of a typical specimen are also shown
in the same figure. The length of the shear span of 1,150 mm and the width of 500 mm of the cantilever part of the specimen were fixed for all the specimens in this program. The effective depth, d, was varied to allow for the variation of shear span-to-depth ratio, a/d, between 3.29 and 6.05. The percentage of steel, p, was also selected to cover the typical range for such structures, as given in Table 1. The end block was designed to simulate the boundary conditions provided by the foundation of typical reinforced concrete bridge piers. The end block provided also an anchorage system for the specimen to the structural floor, and enough anchorage length for the longitudinal reinforcements as shown in Figure 1.

For the two specimens in Series III, the axial compression load was applied using an especially designed load frame equipped with two hydraulic jacks and pin connection to allow for the rotation of the cantilever in the vertical plane, as shown in Figure 2. The hydraulic jacks were connected to an electric pump and pressure regulator to maintain a constant compression load during the test.

All concrete used in fabrication of the specimens was designed for a nominal ultimate strength of 30 MPa and was ready-mixed from a local concrete mix plant. For each specimen, six standard 152.4 x 304.8 mm concrete test cylinders were cast to determine the concrete strength. The reinforcement
used consisted of hot rolled deformed bars graded 300 MPa, conforming to CSA C30-12-72. For each specimen, samples of the steel bar were tested for tension to determine yield stress, ultimate strength, and modulus of elasticity. All samples showed typical ductile behaviour and well defined yield point.

The specimen was fixed to the testing floor using four 31.8 mm diameter high tensile bolts. A bracing system provided by teflon strips was attached at the end of the cantilever to minimize its lateral rotational movement.

TESTING APPARATUS AND PROCEDURE

The vertical reversing loads and deflections were applied using an electric-servo closed loop MTS testing system. The actuator has two universal joints to allow rotation of the actuator and to ensure that the applied load always remains perpendicular to the specimen's surface. The initial reading of the applied load was recorded when the specimen was in the deflected shape under its self-weight.

The specimen was loaded downward (negative load) up to a load equal to the calculated yield load, $P_y$. The corresponding deflection at the location of the load, $\delta_y$, was measured. The specimen was unloaded to the original position and then loaded upward (positive load) to a deflection equal to the yield deflection. The specimen was unloaded and the
same procedure was repeated for two more cycles of deflection reversals. After three cycles, the deflection was increased by increments of $\delta_y$ until the load-resisting capacity of the specimen was less than the yield load, which indicated failure of the specimen.

The load pattern used was similar to that shown in Figure 3. For Series II, the above procedures were used for the first half cycle followed by a fully reversed sine wave loading under the deflection control mode. For Series III, the axial load was applied before the application of the yield load and the measurement of the yield deflection.

The strains of the longitudinal steel were measured using electric resistance strain gauges. The average steel strain was also measured using LVDT's which were mounted on steel studs projecting from the longitudinal reinforcement, as shown in Figure 4. Using the same technique, the average diagonal and vertical strains were measured to determine the average shear strain. Loads and deflections at the end of the cantilever part of the specimen were recorded continuously by an X-Y recorder. Crack propagation was observed using a magnifying lens, and sketches were plotted as the cracks progressed.
TEST RESULTS

Diagonal tension cracks were normally formed as an extension of the initiated flexural cracks under each load direction. Due to the nature of the applied reversed loading condition, the diagonal cracks intersected each other to form an x-shaped diagonal crack for most of the specimens. It was observed in the beginning of the testing program that the hysteresis curves of load-deflection for the second and third cycles were almost the same, as shown in Figure 5 for a typical specimen, 1-4. For this reason and as observed by others (Higashi et al., 1977), a repetition of three cycles for each deflection increment was chosen. A specimen was considered to have failed when its load resisting capacity decreased to less than the calculated yield load, $P_Y$. The ductility factor, $\mu$, was calculated as the ratio between the maximum deflection sustained before failure, $\delta_u$, and the deflection corresponding to the load required to yield the longitudinal reinforcement at the maximum moment section, $\delta_y$. Material properties and the experimental results for the sixteen specimens are given in Table 2. The first digit for the designation of the failure cycle indicates the deflection increment and the final digit refers to the cycle number for that increment. The extent of yielding of the longitudinal reinforcement is also given in Table 2, as a multiple of the effective depth, $d$, of the
corresponding specimen.

The maximum load before failure, $P_{\text{max}}$, given in Table 2 for each specimen is modified to account for the specimen's self-weight. Based on this modified value, the maximum shear stress, $\nu_u$, was calculated for each specimen, as listed in the same Table.

**Crack Configuration**

The typical crack configuration for specimens with a/d values of 3.29 and 4.11 is shown in Figure 6. The failure in these cases occurred mainly due to the widening of the x-shaped diagonal crack. Normally, following this mechanism, a secondary crack parallel to the longitudinal reinforcement will form in the tension side, corresponding to the direction of the failure load.

For specimens with relatively low percentage of steel, i.e., specimens 1-8 and 1-10, the x-shaped diagonal crack was also formed within the d-section. While the width of the diagonal cracks remained very small, the failure was mainly due to the large vertical relative movement at the maximum moment section, as shown in Figure 7. For specimen 1-9, which had a large percentage of steel, the failure was due to the formation of a second diagonal crack, as shown in Figure 8.

For specimens with a/d values of 5.0 and 6.05, the
x-shaped diagonal crack was also formed at a distance approximately d from the maximum moment section. Typical failure occurred due to the initiation of a second diagonal crack at an approximate distance of 2d, as shown in Figure 8. For a specimen with a relatively low percentage of steel in this category, specimen 1-5, the failure was due to local buckling of the longitudinal reinforcement at the location of the x-shaped diagonal cracks, as shown in Figure 9. The formation of a second diagonal crack was not observed.

The observed crack configuration at failure clearly indicates the significant influence of the percentage of steel within a given range of a/d. This confirms the theory that both a/d and p greatly influence the mode of failure, and that a combination of their effects is necessary for the identification of the failure mechanisms for such members.

Strains of the Longitudinal Reinforcements

The measured steel strains indicate typical yielding of the longitudinal reinforcement at the d-section for specimens with a/d values of 3.29 and 4.11. However, for specimens with relatively low percentage of steel in this category, which failed due to large vertical relative movements, no yielding of the longitudinal reinforcement at the d-section was observed before failure. This observation could explain the different
modes of failure observed within this \( a/d \) range.

For specimens with \( a/d \) of 6.05, the measured steel strains generally indicate the yielding of the longitudinal reinforcement at the 2d-section, regardless of the observed crack configuration for specimens with different percentages of steel.

Ductility

The ductility factor, \( \mu \), defined as the ratio between the maximum deflection before failure, \( \delta_u \), and the deflection at yielding, \( \delta_y \), is given for all specimens in Table 2. It is clear that the magnitudes of the ductility factors are relatively low since the specimens were designed to fail in shear. The results indicated that ductility decreases with the increase of the percentage of longitudinal steel, \( p \). It is also clear that the ductility is influenced by the shear span-to-depth ratio, \( a/d \), and the tensile strength of the concrete, \( f_t \), since all the specimens failed in shear (Higai et al., 1982).

The relationship between the ductility factor, \( \mu \), and the above parameters was examined using the experimental data for all the specimens tested in Series I of this program. Based on a multiple linear regression analysis, the following relationship was obtained.
The mean and the standard deviation values, of the ratio of calculated to experimental ductility factor were found to be 1.00 and 0.11 respectively. The tensile strength, $f_t$, could be evaluated based on the measured concrete splitting strength, $f_{sp}$, or the ultimate compressive strength, $f_c'$, as follows (Hwang et al., 1983):

\[ 2a \quad f_t = 0.6 f_{sp} \]

or

\[ 2b \quad f_t = f_c'^{2/3} \]

The power of the different variables included in the parametric relationship, Equation 1, suggests the possibility of using a nondimensional characteristic factor, $K$, to predict the ductility, where

\[ 3 \quad K = \frac{(a/d) f_t}{pf_y} \]

The relationship between the ductility factor, $\mu$, and the proposed characteristic factor $K$, for all the specimens in Series I of this program is shown in Figure 10. Based on a best fitting curve for the experimental data, the following relationship is proposed:

\[ 4 \quad \mu = \frac{1}{0.8-0.13 K} \]

The proposed relationship is shown also in Figure 10 as a solid
The mean and the standard deviation values of the ratio of calculated to experimental ductility factor were found to be 1.00 and 0.09 respectively, clearly indicating a very high degree of predictive accuracy of the proposed expression. Specimens in Series II are also shown in the same figure.

The proposed relation suggests a minimum value of 2.3 for the characteristic factor, K, which corresponds to an integer value for the ductility factor of 2. To exclude the flexural failure mode, the proposed expression provides the maximum value for K of 5.3, which corresponds to a ductility factor of 9.

MODES OF FAILURE

The appearance of the crack configuration discussed previously failed to indicate a clear trend which could be used to classify the mode of failure for such members. The location of yielding of the longitudinal reinforcement was the only common observation which could be used to classify the different mechanisms. This observation confirms the hypothesis that yielding of the longitudinal reinforcement would abrupt the deterioration of the interface shear transfer under the reversed loading condition for members without web reinforcement (Laible, 1977).

Using the proposed characteristic factor, K, it was
possible to classify the modes of failure accordingly, as shown in Figure 10. Based on the range of the $K$ values, the critical crack pattern and the extent of yielding of the longitudinal reinforcement could be predicted. The proposed classification of mode of failure can be summarized as follows:

**Mode (1):** Failure is mainly due to a large vertical relative movement at the maximum moment section, caused by the localization of the yielding of the longitudinal reinforcement at this section. This mode is typical for $K$ values between 4.8 and 5.3.

**Mode (2):** This failure mode is characterized by yielding of the longitudinal reinforcement within a distance $d$ from the maximum moment section. The failure is mainly caused by widening of the $x$-shaped diagonal crack or buckling of the longitudinal reinforcement within the same distance. This mode is typical for $K$ values between 4 and 4.8.

**Mode (3):** The failure is characterized by the yielding of the longitudinal reinforcement within a distance $2d$ from the maximum moment section. Failure is mainly due to the initiation and widening of a second diagonal crack. This mode is typical for $K$-values between 2.3 and 4.

The above ranges of $K$ values corresponding to the three different modes of shear failure were established tentatively and further testing is needed to confirm these
limits, especially in the intermediate range of $K$.

**MAXIMUM SHEAR STRESS**

The intensity of shear stresses is one of the most important parameters that influence the behaviour of bridge piers when they are subjected to large deflection reversals. Increases of the shear stresses cause earlier initiation of the diagonal cracks and increase the pinching phenomenon for the load-deformation hysteresis.

Based on the measured data, the maximum shear stress, $\nu_u$, is normalized with respect to the concrete tensile strength, $f_t$, and related to the characteristic factor, $K$, for all the specimens in Series I, as shown in Figure 11. The solid line represents the best fitting curve for the experimental data, which could be expressed mathematically as follows:

$$[5] \quad \frac{\nu_u}{f_t} = 0.71 \frac{K^{0.74}}{K^{0.74}}$$

The high predictive accuracy of this proposed expression is reflected by the values for the mean and the standard deviation of 0.99 and 0.02, respectively. Specimens in Series II are also shown in the same figure.

The modes of failure were also classified according to the intensity of maximum shear stresses. Using the same specified ranges of $K$ for each failure mode, the ranges of
the intensity of maximum shear stress are established for the three proposed modes, as shown in Figure 11.

The ratio between the maximum shear stress, \( v_u \), and the shear required to yield the longitudinal reinforcement, \( v_y \), for all the specimens in Series I is given in Figure 12. It can be seen that the ratio \( v_u/v_y \) is independent of the percentage longitudinal of steel, \( p \), with a mean value of 1.15.

ENERGY DISSIPATION CAPACITY

In general, the magnitude of the energy dissipation capacity provides a measure for the inelastic performance of a structure under reversed loads. By definition, this energy could be evaluated based on the areas enclosed by the load-deformation hysteresis loops until failure. The relationship between the normalized energy dissipation, \( NED \), with respect to the linearized applied energy of the first cycle, and the ductility factor, \( \mu \), for all the specimens in Series I, is shown in Figure 13. The solid line represents the best fitting curve for the experimental data, which could be mathematically expressed as follows:

\[
NED = 5.75 \mu^{1.67}
\]

The mean and the standard deviations are found to be 1.00 and 0.12, respectively. Thus, for a given ductility, which can be predicted using Equation 4, the normalized energy
dissipation can be determined.

EFFECT OF LOAD FREQUENCY

A total of four specimens, Series II, were tested with different frequencies, as given in Table 1. The behaviour of these specimens was compared to the behaviour of two specimens tested earlier under the very low frequency of 0.0004 Hz, equivalent to a static load in Series I.

The appearance of the crack patterns at failure for the different a/d categories were approximately similar to those specimens tested under an equivalent static load.

Failure of specimens 2-1 and 2-3 occurred due to the widening of the x-shaped diagonal crack just after yielding of the longitudinal reinforcement similarly to specimen 1-1. However, for specimen 2-2, which was tested under the highest frequency, 0.2 Hz, yielding of the longitudinal reinforcement at the d-section was observed at first cycle of the fourth deflection, while the specimen failed during the fifth deflection increment. This observation suggests a possible change in the failure criteria under higher load frequencies. For the a/d value of 6.05, specimen 2-5 failed just after yielding of the longitudinal reinforcement at the 2d-section, similarly to specimen 1-3.

The maximum shear stresses of specimens 2-1, 2-2,
and 2-3 were approximately equal to that of specimen 1-1, as given in Table 2. For the higher range of a/d, the maximum shear stress of specimen 2-5 is slightly higher than that for specimen 1-3. These results reflect the insignificant effect of the load frequency on the maximum shear stress of the specimens in both a/d categories.

Similarly to specimen 1-1, a ductility factor of 4 was obtained for both specimens 2-2 and 2-3. The concrete honeycombs observed at the maximum moment section of specimen 2-1 could contribute to the reduction of the ductility factor obtained for this specimen.

For specimen 2-5 the measured deflection, $\delta_y$, under the same calculated yield load was lower in comparison to the identical specimen 1-3. This could be attributed to the possible torsional movement of specimen 2-5. The change in the yield deflection magnitude, which definitely will alter the ductility, could be the explanation for the two different values obtained for the ductility factors.

EFFECT OF AXIAL LOAD

Two specimens, 3-1 and 3-3, as representative of both the lower and higher ranges of a/d, were subjected to an axial compressive stress of 0.98 MPa and tested in this program. The behaviour of these two specimens is compared
to identical specimens, 1-1 and 1-3 respectively, tested without the presence of the axial load.

Specimen 3-1 failed due to the widening of the x-shaped diagonal crack after yielding of the longitudinal reinforcement at the d-section, similarly to specimen 1-1. As in the case of the identical specimen, 1-3, the failure of specimen 3-3 occurred after yielding of the longitudinal reinforcement at the 2d-section. However, the appearance of the failure in terms of the crack pattern was slightly different. In specimen 1-3, the failure was due to the formation of a second diagonal crack at the 2d-section (failure mode 3). In contrast, the failure of specimen 3-3 was mainly due to the widening of the x-shaped diagonal crack at the d-section (mode 2).

This indicates that the failure mode of the members with large a/d values is significantly affected by the presence of the axial compression stress due to the increase of the maximum shear stress resistance. However, only one specimen was tested in this range and further investigation is required before any general conclusions can be drawn.

SUMMARY AND CONCLUSIONS

The following observations and conclusions are based on the experimental program discussed in this study:

1. The fundamental behaviour of bridge pier structures
can be fully described using a proposed nondimensional characteristic factor, \( K \), in terms of the shear span-to-depth ratio, \( a/d \), concrete tensile strength, \( f_t \), percentage of longitudinal reinforcement, \( p \), and the yield strength of the steel, \( f_y \), as follows:

\[
K = \frac{(a/d) f_t}{p f_y}
\]

2. The proposed factor, \( K \), was used to derive expressions which would predict the ductility, maximum shear stress and energy dissipation capacity for such structures.

3. Using the proposed characteristic factor, \( K \), it was possible to classify the failure mechanisms into three distinct modes of failure.

4. The ratio of the maximum shear stress, \( \nu_u \), to the yield shear stress, \( \nu_y \), is found to be approximately constant and independent of both the percentage of longitudinal reinforcement and the shear span-to-depth ratio. This result indicates the significant effect of yielding of the longitudinal reinforcement on the shear failure under large deflection reversals.

5. The load frequency within the range considered
in this program, 0.0004 to 0.2 Hz, is considered to have an insignificant effect on the mode of failure, maximum shear stress, and ductility. However, the test results indicate possible changes in failure criteria under higher frequencies.

6. The results of the two specimens tested in the presence of the axial compressive load indicated that the axial compressive stresses reduce the deterioration of the interface shear transfer across the crack, which increases the maximum shear stress capacity and ductility of the member.

7. The failure modes of members with large a/d values are affected by the presence of axial compressive stresses.

8. Due to the limited ranges of the load frequency and axial compressive stress in this study, further investigation is required into the effects of values higher than those considered in this program.
REFERENCES


ACKNOWLEDGMENT

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NOTATIONS

\( a \) = shear-span
\( a/d \) = shear-span to depth ratio
\( d \) = effective depth of the section
\( f_{sp} \) = concrete splitting strength
\( f_t \) = concrete tensile strength
\( f_y \) = yield strength of the steel
\( f_{c'} \) = concrete compressive strength
\( K \) = nondimensional characteristic factor
\( \text{NED} \) = normalized energy dissipation capacity
\( P_{max} \) = maximum load in the direction of failure
\( P_y \) = calculated yield load
\( p \) = percentage of longitudinal reinforcement
\( v_c \) = calculated shear strength under monotonic load
\( v_u \) = maximum shear stress
\( v_y \) = shear stress corresponds to yielding of longitudinal reinforcement at maximum moment section
\( \delta_y \) = measured yield deflection
\( \delta_u \) = maximum deflection before failure
\( \mu \) = ductility factor
TABLE 1. Variables Considered in the Experimental Program

<table>
<thead>
<tr>
<th>Series</th>
<th>Spec. No.</th>
<th>Spec. (mm)</th>
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<th>Longitudinal Reinforcement</th>
<th>p%</th>
<th>a/d</th>
<th>$P_y$ (kN)</th>
<th>$V_c$ (MPa)</th>
<th>$V_y$ (MPa)</th>
<th>$\frac{V_c}{V_y}$ Ratio</th>
<th>Frequency of Loading (HZ)</th>
<th>Axial Compression Stress (MPa)</th>
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Figure 1. Relationship Between a Typical Specimen and Bridge Pier

Figure 2. Test Set-up with the Presence of Axial Load
Figure 3. Load Pattern

Figure 4. Instrumentation for Average Steel Strain
Figure 5. Typical Load-Deflection Hysteresis
Figure 6. Final Crack Pattern for Specimen 1-1, $6\delta_y/1$

Figure 7. Final Crack Pattern for Specimen 1-7 at $96\delta_y/3$
Figure 8. Final Crack Pattern for Spec. 1-4, 48_y/1

Figure 9. Final Crack Pattern for Specimen 1-5, 76_y/1
Figure 11. Relationship Between Normalized Shear Resistance and the Characteristic Factor, $K$.

Figure 10. Relationship Between Ductility and the Characteristic Factor, $K$. 

$\mu = \frac{1}{0.8 - 0.13K}$

$\frac{v_u}{f_t} = 0.71 K^{0.74}$
Figure 12. Effect of Percentage of Steel on the Ratio of Maximum to Yield Shear Stress

Figure 13. Relationship Between Normalized Energy Dissipation and the Ductility Factor
| Figure 1. | Relationship between a typical specimen and bridge pier. |
| Figure 2. | Test set-up with the presence of axial load. |
| Figure 3. | Load pattern. |
| Figure 4. | Instrumentation for average steel strain. |
| Figure 5. | Typical load-deflection hysteresis. |
| Figure 6. | Final crack pattern for specimen 1-1, 6δy/1. |
| Figure 7. | Final crack pattern for specimen 1-7, 9δy/3. |
| Figure 8. | Final crack pattern for specimen 1-4, 4δy/1. |
| Figure 9. | Final crack pattern for specimen 1-5, 7δy/1. |
| Figure 10 | Relationship between ductility and the characteristic factor, K. |
| Figure 11 | Relationship between normalized shear resistance and the characteristic factor, K. |
| Figure 12 | Effect of percentage of steel on the ratio of maximum to yield shear stress. |
| Figure 13 | Relationship between normalized energy dissipation and the ductility factor. |