IMPACT TEST ON CONCRETE BRIDGE BARRIERS
REINFORCED WITH GFRP COMPOSITE BARS

FINAL REPORT – Phase III

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ABSTRACT

Corrosion of steel reinforcement is a major cause of deterioration problems in reinforced concrete structures exposed to harsh environmental conditions. The expansion of highway systems increased the need to provide corrosion-free reinforced concrete components for highway bridges. An extensive research program to investigate the behaviour of bridge barriers reinforced with GFRP bars has been going during the last four years at the University of Sherbrooke (Sherbrooke, Quebec, Canada) in collaboration with the Ministry of Transportation of Quebec (MTQ). This program included three phases. The first two phases (Phase I and Phase II) involved the behaviour of barriers under static loading conditions. The third phase (Phase III) involved the behaviour of barriers subjected to impact loads. The geometry, concrete dimensions and reinforcement of both PL2 and PL3 barriers were based on the new Canadian Highway Bridge Design Code (CHBDC, 2000). This report presents the results of the impact test; phase III, which was carried out in the field. A total of 8 full-scale 10-m long barriers prototypes were constructed and tested. The tests included 4 PL2 and 4 PL3 prototypes. For each type of barriers, two prototypes were reinforced with GFRP ISOROD bars and the other two were reinforced with steel bars. For both types of reinforcement, GFRP and steel, a new detailing for connecting the wall to slab by extending the main reinforcement of the wall through the slab was introduced. Pendulum crash tests using a 3.0-ton pear-shaped iron ball were performed under the same conditions for each type of barriers, PL2 or PL3, to compare the behaviour of barriers reinforced with GFRP rods to their counterparts reinforced with steel. Based on the results of this investigation, it is concluded that the behaviour of PL2 and PL3 bridge barriers reinforced with GFRP ISOROD bars is very similar to their counterparts reinforced with conventional steel in terms of cracking, energy absorption and ultimate strength.
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1. INTRODUCTION

In the recent years, the advanced composite materials have been used in many structural systems for civil engineering applications especially those in high corrosive environments such as marine structures, bridge decks and superstructures exposed to deicing salts as well as structures supporting equipment sensitive to electromagnetic fields. The noncorrosive and nonmagnetic properties of FRP reinforcement are significantly beneficial in such structures.

The expansion of the highway systems increased the need to provide virtually maintenance-free reinforced concrete components for highway bridges. Maheu and Bakht (1994) have developed a new design for bridge deck overhangs and barrier walls using glass FRP NEFMAC grids. This new barrier wall, referred to as Ontario Bridge Barrier, is connected to the deck slab by means of double-headed tension bars in a single row. As a part of the continuing effort to achieve this goal, a new set of tests on concrete bridge barriers reinforced with GFRP bars were carried out. A new reinforcement detailing using GFRP rods for PL2 and PL3 concrete barriers has been developed. The geometry, concrete dimensions and amount of reinforcement are based on the new Canadian Highway Bridge Design Code (CHBDC, 2000).

A new corrosion-free connection between the slab and the barrier wall is proposed. In which the main reinforcement of the wall continues all the way through the slab thickness. However, in the current reinforcement detailing according to the provisions of
Section 16.10 in the new CHBDC (2000), the proposed connection by Maheu and Bakht (1994) using double-headed tension bars of steel at 300 mm centers, is adopted.

It should be noted that in the new CHBDC (2000), it is allowed to use only one planar of FRP grid, or one orthogonal assembly of FRP bars near the tensile face with fibre reinforced concrete (FRC) comprising low-modulus polymer fibres. However, the use of FRC is not required if the barrier wall is also provided with an FRP grid or an orthogonal assembly of FRP bars near the compression face.

To investigate the validity of the proposed design using GFRP bars as reinforcement for the barrier wall and the connection between the wall and the slab, an elaborated test program has been carrying out at the University of Sherbrooke in collaboration with the Ministry of Transportation of Quebec and Pultrall Inc. (Thetford Mines, Quebec) in the last four years. The test program consists of three phases. The first two phases investigated the behaviour of barriers under static loading conditions, while the third phase included the impact test. For all barriers (types PL2 and PL3), the reinforcing bars have four different shapes with the implementation of the new detailing for connecting the wall to the slab. Three shapes D1, D2, and P1 were used for vertical reinforcement as shown in Figures 1 and 2. The fourth shape, H1, was a straight bar for horizontal reinforcement. In phase I, two reinforcement configurations were used. In configuration 1, all shapes of reinforcing bars were made of steel while in configuration 2, three shapes D1, P1 and H1 were made of GFRP bars and shape D2 only was made of steel. A total of 4 PL2 and 6 PL3 full-scale barrier prototypes were tested (Masmoudi et.
For each type of barriers, PL2 and PL3, two identical prototypes were constructed for each reinforcement configuration as well as two PL3 barriers using reinforcement configuration 1, with all shapes of reinforcing bars made of size 15M bars. Very similar behaviour for the ten tested barriers was obtained. Thus, it was decided, in phase II, to test barriers totally reinforced with GFRP bars. In phase II, a total of 6 full-scale barrier models were constructed and tested (Masmoudi et. al., 2001). The tests included 3 PL2 and 3 PL3 prototypes, two of each were reinforced with GFRP C-BAR reinforcing rods (manufactured by Marshall Industries Composites Inc., Lima, Ohio, USA) and the third one was reinforced with GFRP ISOROD bars (manufactured by Pultrall Inc., Thetford Mines, Québec, Canada). Results indicate that the static behaviour and the ultimate capacity of PL2 and PL3 concrete barriers reinforced with GFRP are similar to their counterparts reinforced with conventional steel. As well, the behaviour of the identical barriers reinforced with the two types of GFRP, C-BAR and ISOROD was very similar. The ultimate capacities of the barriers were compared to the equivalent ultimate static capacities of PL2 and PL3 barriers reinforced either with GFRP or steel bars as calculated following the procedures of the AASHTO (1994) LRFD Bridge Design Specifications (see Appendix A). The test results were in good agreement with the values permitted by AASHTO (LRFD Bridge Design Specifications, 1994). No failure occurred at the base of the wall where the new connection between the wall and the slab exists.

The new Canadian Highway Bridge Design Code (CHBDC, 2000) and the AASHTO (Guide Specifications for Bridge Railing, 1889) require that the wall barrier should have sufficient strength to survive the initial impact of a collision and to remain
effective in redirecting the vehicle. Both Codes specify that the strength of the barrier should be determined from a crash test using a designated vehicle. Thus, phase III of the study included performing the impact tests on the barriers. To verify the new detailing for the connection between the wall and the slab as well as the validity and the effectiveness of using GFRP reinforcing bars as reinforcement for barrier walls, a pendulum crash test was carried out. As there are no specifications available for the pendulum crash test, it was decided to perform the test on barriers reinforced with GFRP ISOROD bars as well as on identical barriers reinforced with conventional steel under the same conditions. It should be noted that the pendulum crash test described herein is different from the conventional pendulum tests (Scanlon et al., 1989) as it is calibrated against identical barriers reinforced with steel bars. The undertaken test herein is similar to the pendulum test carried out on the Ontario Bridge barriers by Klement and Aly (1998). A comparison of the behaviour of barriers reinforced with GFRP and conventional steel is valid, as the performance of steel reinforced concrete barriers has been verified by long-term service in the field.

This report presents the results of the pendulum crash tests, phase III of the study, that were carried out to investigate the behaviour of both types PL2 and PL3 concrete barriers reinforced with GFRP bars compared to that of steel reinforced ones under impact load.
2. TEST PROGRAM

2.1 Test prototypes

A total of eight full-scale 10-m long reinforced concrete barriers were constructed and tested under impact load. The barriers were divided into two series. Each series contains 4 identical barriers, two reinforced with conventional steel bars and the other two reinforced with GFRP ISOROD bars. Series PL2 contains barriers PL2-ST1, PL2-ST2, PL2-IS1 and PL2-IS2 for barriers meeting performance level 2. Series PL3 contains barriers PL3-ST1, PL3-ST2, PL3-IS1 and PL3-IS2 for barriers meeting performance level 3.

The barriers’ designations can be explained as follows. The first three characters PLX denote the performance level (PL), where X equals 2 or 3 as introduced in the new CHBDC (2000). The following two characters indicate the type of reinforcement, ST for steel bars and IS for ISOROD GFRP bars as well as the number 1 or 2 indicates the prototype number.

The geometry, concrete dimensions and the amount of reinforcement based on the new Canadian Highway Bridge Design Code (CHBDC, 2000) for the barriers of Series PL2 and PL3 are shown in Figures 1 and 2, respectively. The height and the breadth of the base of the wall are 880 mm and 410 mm for PL2 and 1140 mm and 435 mm for PL3, respectively. For all shapes of reinforcing bars of PL2 barriers, 15M (200 mm²) steel bars and #5 (198 mm²) GFRP ISOROD bars were used. For PL3 barriers, reinforcing bars of
shapes D1 and H1 were either 15M (200 mm²) steel bars or #5 (198 mm²) GFRP ISOROD bars while shapes D2 and P1 were either 20M (300 mm²) steel bars or #6 (285 mm²) GFRP ISOROD bars. For both types of barriers, the vertical spacing between bars D1, D2, and P1 was 200 mm, while the horizontal spacing between bars H1 was 181 mm and 141 mm for PL2 and PL3 barriers, respectively.

The prototypes were cast-in-place on two stages. Stage I included casting an 11.0 m long, 1.5 m wide and 0.25 m thick slab on a horizontal bed made of concrete and covered with plastic sheets to prevent bond between the bed and the slab. The slab was reinforced with two steel layers. Each layer has 15M bars @ 100 mm in the short direction and 15M bars @ 200 mm in the long direction, as shown in Fig. 3. Shapes D1 and D2 reinforcement (Fig. 1 and 2) were installed in position and tied to the slab cages. Also, two horizontal straight bars, H1, were tightened at the top of D1 and D2 reinforcement as a distributor to keep the desirable spacing among them. 100-mm diameter holes through the slab thickness were obtained by installing 100-mm diameter PVC tubes in the slab, thus to enable anchoring the slab to the rigid flooring system during testing. The holes located at both sides of the wall with 0.5 m spacing. In stage II, after curing the slab for few days, the longitudinal and vertical reinforcement (shape P1) were added and the wall was cast. Figures 3 through 10 show the different stages of barrier construction. The first barrier was cast on 14th of November, 2000 and the last one on January 9, 2001. Table 1 lists the time schedule for casting the slab and the wall for each barrier. It should be noted that all curing procedures were according to the MTQ’s specifications where the temperatures of barriers kept above 10 °C during the first week of curing. The barriers
cast before 1\textsuperscript{st} of December were covered with 25-mm thick insulation sheets (see Fig. 11). While the barriers cast after December 1\textsuperscript{st} were cured in a temperature-controlled tent built arround the barrier using gas-heat (see Fig. 12).

2.2 Material properties

The barriers were constructed using a ready-mixed normal weight concrete (concrete Type V, MTQ). Table 2 lists the concrete strengths based on the average value from the compression tests performed on at least three $150 \times 300$ mm cylinders for each concrete batch. The tensile strength of concrete was determined by performing the split cylinder tests. The properties of the second generation of the GFRP ISOROD bars (composed of 75\% type-E glass fibres and 25\% Vinylester resin) were determined by performing tensile tests on straight specimens cut from the actual bent bars as shown in Figures 13 and 14. Figures 15 and 16 show the test set-up for the tensile test and the mode of failure of the GFRP rods, respectively. The properties of the GFRP ISOROD bars as well as the steel used in reinforcing the barriers are listed in Table 3.

2.3 Instrumentation

For each barrier, a total of 26 electrical resistance 5-mm long strain gauges were used to measure strains in reinforcing bars at critical locations. The gauges were glued to shapes D1, D2 and P1 reinforcement at 3 different sections at the middle part of the wall as well as to shape H1 at 4 different locations in the front and back layers as shown in Figure 17. At each section, 6 strain gauges were installed in the positions shown in Figure 18. An accelerometer was mounted on the back (compression) face of the wall directly
behind the point of strike to measure the peak and the duration of the impact load. Also, the crack widths on both faces of the wall, directly after each test, were measured and recorded. A very high speed data acquisition system (5000 reading/s), provided by the Ministry of Transportation of Quebec, was used to collect and record the data from the accelerometer and the strain gauges.

2.4 Test Set-up and Procedure

At the site of testing, a big hole, 14.0 m (length) × 4.0 m (width) × 1.0 m (depth), was digged out to build a rigid reinforced concrete test bed. The test bed was 12.0 m long, 2.0 m wide and 1.0 m thick. A total of 24, 32-mm diameter, Dywidag bars were embedded vertically in the rigid test bed acting as anchors (dowels) with 0.95 m embedment length and 0.50 m free length (Fig. 19). The spacing among these anchor bars is 1.0 m and they were positioned such that they fit every other hole in the slab of the barrier. The top surface of the rigid bed was leveled with the ground surface on site. The barrier was carried to the test bed by means of trailer using 50-ton mobile crane for loading and unloading. The crane lifted the barrier at four locations, which were predetermined such that no cracks occur in the barrier during handling (Fig. 20).

The barrier was adjusted in position where the 32-mm Dywidag bars (anchors) went through their corresponding holes in the slab. Then, the barrier was tightened to the rigid test bed by nuts, as shown in Fig. 21. 45-mm thick square steel plates (200 × 200 mm) were used as bearing plates between the nuts and the slab.
The pendulum impact test was carried out using a pear-shaped iron ball having a weight of 3.0 ton. The iron ball was suspended from the boom of another 80 ton mobile crane at a height of approximately 20 m from the test bed. The crane is positioned such that the first point on the ball that hits the wall is at 0.75 m and 0.90 m above the base of the wall for PL2 and PL3 series, respectively. The ball was pulled lateraly to the desirable height with the help of a pulley system powered by the same crane. Both cables supporting and holding the iron ball in position were at the same plane, which was perpendicular to the face of the wall at the targeted point of strike on the wall. Figure 22 shows a schematic diagram for the test set-up. The height of the wall was established by using a survaying instrument, which can accurately determine the difference in level between the point on the iron ball that first hits the wall and the targeted strike point on the wall. During the test, the ball was released, after establishing the required height, using the same pulley system that was used to lift it. Figure 23 shows a photo for the test set-up.

2.5 Calibration of Pendulum Test

If the barriers reinforced with GFRP bars, which have identical geometry could sustain with the same or less amount of damage the impact of the same ball swung from the same height, then it can be stated with confidence that those barriers have the same strength as the well-established ones reinforced with conventional steel.

To establish the adequate height of the wrecking ball along with a configuration for transmitting the impact of the ball to the wall that cause a considerable but not extensive damage to the barrier, several attempts with different configurations were carried out on
the barriers reinforced with conventional steel. After each attempt, the amount of damage received by the barrier wall was evaluated.

The first configuration was done by transferring the impact of the ball to the barrier through a combination of two steel plates and two truck tires, which were mounted on rims and inflated to a pressure of approximately 0.6 MPa, as shown in Fig 24. The two tires were rested on the wall, directly adjacent to each other, and were raised on timber backing such that they touch the wall near the top as well as near the bottom. The tires, then, were covered by two steel plates of 25-mm thickness each. The targeted strike point was marked on the outer steel plate at the center line between the two tires. This arrangement was done to simulate the peak impact caused by a vehicle tractor rear axle assembly in a crash. The first strike was done on specimen PL2-ST1 at a location 1.25 m from the edge of the wall, designated "location (1)". The iron ball was raised to a height of 3.0 m and was swung into the target. The impact did not displace or cause any damage to the barrier. The test was repeated at the same location, (1), by swinging the ball from a height of 3.5 m, which is the maximum height deliverable by the equipment in hand. Same results were obtained. So, it was decided to have another configuration for effectively transmitting the impact of the ball to the wall.

The second configuration was done by using only the two steel plates without the tires, which absorbed a considerable amount of the impact energy. The steel plates were directly supported on the slab and touch the wall at the bottom (at the top of the vertical part of the wall) as well as at the top as shown in Fig. 25. The same test procedure was
repeated (strike number 3) at the same location (1) by swinging the ball from a height of 3.0 m. The barrier end section was displaced and the concrete was crashed locally at the lines of contact with the steel plates (see Figure 26).

The third configuration was done also by using the steel plates only. The steel plates were raised on timber filling to be in direct contact, through their whole area, with the sloping face of the wall to prevent any localized premature failure (Fig. 27). The test was repeated (strike number 4) at the same location (1) by swinging the ball from a height of 3.0 m. The impact of the wall dislodged a 2.9 m long wide segment of the wall. The damage received by the barrier was not extensive, but it was considered sufficient for the purpose of comparison. To confirm these results, the test, using the third configuration, was repeated at the other end of the barrier (strike number 5) at a distance of 1.25 m from the other edge of the wall, designated "location 2" (see Figures 28 and 29). Similar results to those of strike number 4 were obtained. Thus, it was decided, for barriers of series PL2, to use that third configuration as shown in Fig. 30, placing the two steel plates in full contact with the wall, and swing the ball from a height of 3.0 m.

For barriers of series PL3, the same steel plates' configuration was used as in Series PL2, but the ball was raised to the maximum height of 3.5m. Two tests were carried out on specimen PL3-ST1 at its two ends, 1.25 m from the edge. Similar results for both stikes were obtained as shown in Figures 31 and 32. The damage received by the barrier was not extensive, but again it was considered sufficient for the purpose of comparison. Thus, it was decided, for barriers of series PL3, to use the same configuration as Series
PL2, but swinging the ball from a height of 3.5 m. The results of all stikes, at the edge of
the walls, were not considered in the final results. Only results from striking the middle
part of the barrier were considered.

3. TEST RESULTS AND DISCUSSION

The initial energy delivered by the wrecking ball to the wall barriers is 86818 J and
101288 J for the strike heights of 3.0 m (PL2-barriers) and 3.5 m (PL3-barriers),
respectively. This potential energy, which equal to the kinetic energy is calculated
according to the relation:

\[ m \cdot g \cdot h = 0.5 \cdot m \cdot v^2 \]

Where, m is the mass of the iron ball (2950 kg), g is the gravity acceleration (9.81
m/s$^2$), h is the height of the ball (m), and v is the velocity of the ball. This initial energy of
the impact ball is used to deform the barrier wall during the impact. When the barrier wall
reaches its maximum displacement, the velocity of the impact becomes zero and all the
initial energy is used to deform the barrier assuming that the energy loss due to
deformation of the impact iron ball and the steel dywidag anchors is negligible. The
stresses developed in the barrier at the first moment of strike consist of two parts: the
impact force directly induced by the wrecking ball, the other is the inertia force produced
by the vibration of the wall. After the first moment of strike, the stresses developed in the
barrier were only caused by the inertia force. The impact and inertia forces depend on the
structural stiffness, the higher the stiffness, the larger the impact and inertia forces are. If
the barrier cannot absorb that much energy, the impact energy is released through
cracking and deformation of the barrier. For barriers subjected to the same impact energy, the residual (unrecovered) deformation depends on the properties of the materials, stiffness of the wall, and the distribution and width of cracks. Due to the low stiffness of GFRP composites, larger number of cracks and crack widths should be expected in the barriers reinforced with GFRP bars than those of their counterparts reinforced with conventional steel.

The measured acceleration of the ball, at the moment it stroke the wall, was used to calculate the impact force. Tables 4 and 5 list the values of the calculated impact forces for PL2 and PL3 barriers, respectively. It should be noted that, for barriers PL2-IS2 and PL3-ST2, the small values for the measured acceleration and consequently for force and energy, compared to their counterparts at the same ball height, may be attributed to an uneven torque, which was applied to tighten the slab of the barrier to the test bed and was not calibrated or measured. This uneven tightening led to different energy consumption that was necessary to move the barrier as a result of the impact load. Also, the close values of measured acceleration for the two heights, 3.0 m and 3.5 m can be similarly explained. Figures 33 and 34 show the typical relationship for the measured acceleration versus time for PL2 and PL3 barriers, respectively. It can be seen that the impact effect lasts for 0.1 S and reached its peak value during the first 0.03 S.

The test results will be presented in terms of cracks pattern and strains in the reinforcing bars.
3.1 PL2 Barriers

3.1.1 Cracking and General Behaviour

Figures 35 through 38 show schematic drawings for the cracks pattern on the front face of the PL2 barriers. All PL2 barriers, either reinforced with GFRP or steel bars, behaved in a similar manner. The barriers suffered from diagonal cracks symmetrically distributed around the vertical axis, which passes through the point of strike on the wall. These cracks spread over a length, defined by AASHTO as the critical length, of 2.9 m to 3.3 m at the top of the wall coming down to 0.9 m to 1.2 m at the base of the wall. The measured critical lengths are in good agreement with the values predicted by the yield line approach (2.90 m and 2.95 m for steel and GFRP reinforced barriers, respectively) developed by Hirsch (1978) and later adopted by AASHTO LRFD Bridge Design Specifications (1994).

For barriers PL2-IS1 and PL2-IS2 reinforced with GFRP bars, the number of cracks was larger but the spacing between these cracks was smaller than those of their counterparts reinforced with steel. For barriers PL2-ST1 and PL2-ST2, the maximum measured crack widths on the front face were 0.65 mm and 0.45 mm while these values were 0.70 mm and 0.46 mm on the back face of the wall. On the other hand, for barriers PL2-IS1 and PL2-IS2, the maximum measured crack widths on the front face were 0.75 mm and 0.85 mm while these values were 0.66 mm and 0.90 mm on the back face of the wall. The average of the maximum measured crack widths for the two barriers reinforced with GFRP bars were 1.45 and 1.35 times larger than those of the barriers PL2-ST1 and PL2-ST2 reinforced with steel bars on the front and back faces, respectively. Table 6 lists
the maximum measured crack widths on the front and back faces of the tested PL2 barriers.

It should be noted that the two vertical cracks at the mid-section of PL2-IS2 barrier wall were initially developed before testing due to temperature variation during curing of the concrete.

3.1.2 Strain in Reinforcing Bars

Figures 39 through 42 show the strain gauges readings versus time for horizontal and vertical reinforcement of PL2 barriers. It can be noticed that the strains in the horizontal reinforcement were negligible in the barriers reinforced with steel compared to those in the barriers reinforced with GFRP bars, which reached 5500 με for PL2-IS2. However, strains in the vertical reinforcement were higher in the barriers reinforced with steel compared to those in the barriers reinforced with GFRP bars. This indicates that due to the high stiffness of the steel reinforced barriers most of the impact force was carried by the vertical steel reinforcement, which was closer to the concrete surface. While, for GFRP reinforced Barriers, with lower stiffness, the horizontal reinforcement has more contribution in carrying the impact load.

Figures 43 to 48 show the final cracks pattern on the front and back faces of PL2 barriers.
3.2 PL3 Barriers

3.2.1 Cracking and General Behaviour

Figures 49 through 52 show schematic drawings for the cracks pattern on the front face of the PL3 barriers. All PL3 barriers, either reinforced with GFRP or steel bars, behaved in a similar manner identical to PL2-barriers. The diagonal cracks spread over critical lengths of 3.7 m to 4.1 m at the top of the wall coming down to 1.0 m to 1.3 m at the base of the wall. Also, these measured critical lengths are in good agreement with the values predicted by the yield line approach (4.4 m and 4.6 m for steel and GFRP reinforced barriers, respectively).

For barriers PL3-IS1 and PL3-IS2 reinforced with GFRP bars, the density of cracks and the crack widths were very similar to those of their counterparts reinforced with steel. For barriers PL3-ST1 and PL3-ST2, the maximum measured crack widths on the front face were 0.60 mm and 0.56 mm while these values were 0.45 mm and 0.65 mm on the back face of the wall. On the other hand, for barriers PL3-IS1 and PL3-IS2, the maximum measured crack widths on the front face were 0.54 mm and 0.80 mm while these values were 0.55 mm for both barriers on the back face of the wall. The average of the maximum measured crack widths for the two barriers reinforced with GFRP bars were very much the same as those of the barriers PL3-ST1 and PL3-ST2 reinforced with steel bars (1.0 to 1.15 times larger). This may be due to the smaller spacing between the horizontal bars than that of type PL2 barriers. Table 7 lists the maximum measured crack widths on the front and back faces of the tested PL3 barriers.
Similar to barrier PL2-IS2, the two vertical cracks at the mid-section of PL3-IS2 barrier wall were developed before carrying out the impact test due to temperature variation during curing as mentioned before.

### 3.2.2 Strain in Reinforcing Bars

Figures 53 through 56 show the strain gauges readings versus time for horizontal and vertical reinforcement of PL3 barriers. The general strain behaviour of the reinforcement bars was very similar to PL2 barriers. Figures 57 to 64 show the final cracks pattern on the front and back faces of PL3 barriers.

For sake of comparison, figures 65 and 66 show, on one page, the cracks pattern on the front face of PL2 and PL3 barriers, respectively,

Two interesting tests were carried out on barriers PL3-ST2 and PL3-IS1 at the designated location (1) near the edge of the wall by striking the barrier with the ball directly without using the steel plates as shown in figure 67. The same ball height of 3.5 m was used. For both barriers, the impact of the ball caused dislodging of big concrete pieces as shown in Figures 68 through 73. For barrier PL3-ST2, both horizontal and vertical reinforcement were completely bent and damaged as shown in Figure 72. However, for barrier PL3-IS1, both horizontal and vertical GFRP bars were still straight as shown in Figure 73.
CONCLUSIONS

This report presents the results of phase III of a test program that has been carried out, for the last four years, at the University of Sherbrooke in collaboration with the Ministry of Transportation of Quebec and Pultrall Inc. (Thetford Mines, Quebec) to develop a new design for a maintenance-free bridge barriers and validate the use of FRP bars as a feasible reinforcement material to achieve such a goal. In this phase of study, pendulum impact tests, using a 3-ton wrecking iron ball, were carried out on both PL2 and PL3 bridge barriers. The main objective of these tests is to compare the general behaviour and cracking patterns of barriers reinforced with GFRP bars to those of identical geometry reinforced with conventional steel. Furthermore, a new connection detailing between the wall and the slab was investigated. The geometry, concrete dimensions and amount of reinforcement of the tested barriers are based on the new Canadian Highway Bridge Design Code (CHBDC, 2000). Normal weight concrete Type V (MTQ) along with two assemblies of orthogonal GFRP bars, near the tension face as well as near the compression face of the wall, were used. A new connection between the slab and the barrier wall is proposed, in which the main reinforcement of the wall continues all the way through the slab thickness using GFRP bent bars. The pendulum test was calibrated such that the impact force caused a noticeable but not extensive damage to the barrier wall. Based on the test results, the following conclusions can be drawn:
1. The behaviour of bridge barriers of both types, PL2 and PL3, reinforced with GFRP bars, subjected to impact load, was very similar to that of the identical barriers reinforced with conventional steel.

2. For type PL2 barriers reinforced with GFRP bars, although the stiffness of the GFRP bars are only 1/5 that of the steel bars, the maximum measured crack width on the front face of the barrier wall was 0.8 mm, which represents 1.45 times larger than that measured for their companions reinforced with steel.

3. For type PL3 barriers reinforced with GFRP bars, the crack patterns and the crack widths were very similar to those of their counterparts reinforced with steel. The maximum measured crack width on the front face of the barrier wall was 0.67 mm, which represents 1.15 times larger than that measured for their companions reinforced with steel. This may be due to the better arrangement of the horizontal reinforcement with less spacing in this type of barriers than in type PL2 barriers.

4. The new connection between the slab and the barrier wall sustained and transferred the impact load successfully without showing any sign of failure.

5. In barriers reinforced with GFRP bars, the horizontal reinforcement contributed more, in carrying the impact load, than that of barriers reinforced with steel. This also, may be due to the higher stiffness of steel, which allowed the absorption of the impact energy by the vertical reinforcement closet to the impact load effect.
RECOMMENDATIONS

Based on the observed behaviour and the impact test results of PL2 and PL3 barriers reinforced with GFRP bars, the following recommendations can be stated:

1. PL2 and PL3 barriers reinforced with GFRP bars, with the reinforcement details as shown in Figures 1 and 2, respectively, can be used safely, as a maintenance-free bridge element, instead of the conventional steel barriers. The new CHBDC (2000) allows the use of normal weight concrete (Type V, MTQ was used in this study) if two assemblies of orthogonal GFRP bars, near the tension face as well as near the compression face of the wall are used (two layers of GFRP ISOROD were used in this study). For all shapes of vertical reinforcing bars of PL2 barriers, #5 (198 mm$^2$) GFRP ISOROD bars were used. For PL3 barriers, reinforcing bars of shapes D1 and H1 were #5 (198 mm$^2$) GFRP ISOROD bars while shapes D2 and P1 were #6 (285 mm$^2$) GFRP ISOROD bars. For both types of barriers (PL2 and PL3), the vertical spacing between bars of all shapes (D1, D2, and P1) was 200 mm, while the horizontal spacing between horizontal bars (H1) was 181 mm and 141 mm for PL2 and PL3 barriers, respectively.

2. The amount of reinforcement shown in Figures 1 and 2 for PL2 and PL3 barriers, respectively, should be doubled at the end of the barrier wall for the last 5.0 m as described by CHBDC (2000).

3. The amount of horizontal reinforcement for type PL2 barriers may be increased. Spacing between horizontal bars of 140 mm, as in PL3 barriers, seems to be adequate for cracking control.
ACKNOWLEDGMENT

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REFERENCES


**Table 1** Time schedule for casting the barriers

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Table 3 Properties of steel and GFRP reinforcing bars

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Table 4 Maximum measured accelerations and impact loads for PL2 barriers

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Table 5 Maximum measured accelerations and impact loads for PL3 barriers

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Table 6 Maximum measured crack widths, (mm), for PL2 barriers

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Table 7 Maximum measured crack widths, (mm), for PL3 barriers

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<td>Back Face Max.</td>
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Figure 1. Dimensions and reinforcement details for PL-2 barriers.
Figure 2. Dimensions and reinforcement details for PL-3 barriers.
Figure 3. Reinforcement of the slab and the first part of the wall (PL3-ST)

Figure 4. Pouring concrete into the slab of the barrier (PL2-IS)
Figure 5. Finishing the slab surface (PL3-ST)

Figure 6. Complete wall reinforcement (PL3-IS)
Figure 7. Installing formwork sides of the wall (PL2-ST)

Figure 8. Finishing the top surface of the wall (PL3-IS)
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Figure 10. A barrier wall after painting (PL3)
Figure 11. A barrier wall covered with insulation sheets

Figure 12. A barrier wall during curing in winter
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Figure 16. Failure of GFRP test specimens
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Figure 18. Typical positions of strain gauges at each wired section in the wall.
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(b) After Construction

Figure 19. The concrete rigid test bed.
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**Figure 21.** Dywidag anchor system between the slab of the barrier and the test bed.
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Figure 23. Photo for the test set-up (barrier PL2-ST1)
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Figure 25. Steel plates directly supported on the slab of the barrier.
Figure 26. Steel plates raised on timber filling.

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Figure 32. Back view of barrier PL3-ST1 after the first strike
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Figure 34. Typical acceleration – time relationship for PL3 barriers.
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Figure 36. Schematic drawing for cracks pattern on the front face of barrier PL2-ST2

Figure 37. Schematic drawing for cracks pattern on the front face of barrier PL2-IS1

Figure 38. Schematic drawing for cracks pattern on the front face of barrier PL2-IS2
Figure 39. Strains versus time relationship for barrier PL2-ST1
Figure 40. Strains versus time relationship for barrier PL2-ST2
Figure 41. Strains versus time relationship for barrier PL2-IS1
Figure 42. Strains versus time relationship for barrier PL2-IS2
Figure 43. Cracks pattern on the front face of barrier PL2-ST1

Figure 44. Cracks pattern on the back face of barrier PL2-ST1
Figure 45. Cracks pattern on the front face of barrier PL2-ST2

Figure 46. Cracks pattern on the front face of barrier PL2-IS1
Figure 47. Cracks pattern on the front face of barrier PL2-IS2

Figure 48. Cracks pattern on the back face of barrier PL2-IS2
Figure 49. Schematic drawing for cracks pattern on the front face of barrier PL3-ST1

Figure 50. Schematic drawing for cracks pattern on the front face of barrier PL3-ST2

Figure 51. Schematic drawing for cracks pattern on the front face of barrier PL3-IS1

Figure 52. Schematic drawing for cracks pattern on the front face of barrier PL3-IS2
Figure 53. Strains versus time relationship for barrier PL3-ST1
Figure 54. Strains versus time relationship for barrier PL3-ST2
Figure 55. Strains versus time relationship for barrier PL3-IS1
Figure 56. Strains versus time relationship for barrier PL3-IS2
Figure 57. Cracks pattern on the front face of barrier PL3-ST1

Figure 58. Cracks pattern on the back Face of barrier PL3-ST1
Figure 59. Cracks pattern on the front face of barrier PL3-ST2

Figure 60. Cracks pattern on the back face of barrier PL3-ST2
Figure 61. Cracks pattern on the front Face of barrier PL3-IS1

Figure 62. Cracks pattern on the back face of barrier PL3-IS1
Figure 63. Cracks pattern on the front face of barrier PL3-IS2

Figure 64. Cracks pattern on the back face of barrier PL3-IS2
Figure 65. Cracks pattern on the front face for all PL2 barriers
Figure 66. Cracks pattern on the front face for all PL3 barriers
Figure 67. Iron ball impact the barrier without steel plates
Figure 68. Front view of barrier PL3-ST2 (Strike without steel plates)

Figure 69. Back view of barrier PL3-ST2 (Strike without steel plates)
Figure 70. Front view of barrier PL3-IS1 (Strike without steel plates)

Figure 71. Back view of barrier PL3-IS1 (Strike without steel plates)
Figure 72. Close up to the steel reinforcement of barrier PL3-ST2

(Strike without steel plates)

Figure 73. Close up to the steel reinforcement of barrier PL3-IS1

(Strike without steel Plates)
Appendix A

STRENGTH OF BRIDGE WALL BARRIERS

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS -1994
The Barrier shall have sufficient strength to survive the initial impact of the collision and to remain effective in redirecting the vehicle.

The assumed yield line pattern (Hirsch, 1978) caused by a truck collision that produces a force $F_t$ that is distributed over a length $L_t$ is shown in Fig. 1. The following expressions are developed for the strength of the barrier with uniform thickness, which based on the formation of the yield lines (Fig. 1) at the limit state.

\[
R_w = \left(\frac{2}{2L_t - L_t}\right) \left(8M_b + 8M_wH + \frac{M_cL^2_c}{H}\right)
\]

\[
L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_wH)}{M_c}}
\]

Where,
- $R_w =$ Barrier nominal resistance to transverse load
- $M_b =$ moment strength of beam at top (if any)
- $M_w =$ moment strength of wall about vertical axis
- $M_c =$ moment strength of wall about horizontal axis
- $H =$ height of wall
- $L_t =$ longitudinal distribution length of impact force
- $L_c =$ critical wall length of yield line pattern

For barrier walls having sloping faces, the recommended procedure is to use the same equations with average values for moments $M_w$ and $M_c$. The $R_w$ calculated by using average values was 4% less (conservative) than the more exact approach. The forces that must be resisted by the barrier for PL-2 and PL-3 are given in Table 1.

Table 1. Design Forces

<table>
<thead>
<tr>
<th>Direction</th>
<th>PL-2</th>
<th>PL-3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Force (KN)</td>
<td>Length (mm)</td>
</tr>
<tr>
<td>Transverse</td>
<td>240</td>
<td>1070</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>80</td>
<td>1070</td>
</tr>
<tr>
<td>Vertical</td>
<td>80</td>
<td>5500</td>
</tr>
</tbody>
</table>
Figure A-1. Yield Line Pattern
(1) **PL2 BARRIERS**

### (1.1) STEEL – REINFORCED BARRIERS

- $\phi = 1.0$ (resistance factor)
- $f_s = 400$ MPa
- $f_c = 35$ MPa
- $A_s (15M) = 200$ mm$^2$

**Step (1): $M_w$ about vertical axis** (Fig. 2)

#### Segment I  (neglect $A_s'$)

- $A_s = 3 \times 200 = 600$ mm$^2$
- $d_{2v} = 98 + 63 = 161$

$$a = \frac{A_s f_s}{0.85 f'c} = \frac{600 \times 400}{0.85 \times 35 \times 555} = 14.54 \text{ mm}$$

$$\phi M_{nl} = \phi \times A_s f_s \left( d - \frac{a}{2} \right)$$

$$\phi M_{nl} = 1.0 \times 600 \times 400 \left( 161 - \frac{14.54}{2} \right)$$

$$M_{nl} = 36.895 \times 10^6 \text{ N mm}$$

#### Segment II

- $A_s = 0$

$$\phi M_{nl} = 0.0$$
Figure A-2. PL2 Barriers
**Segment III**

\[ M_{n+ve} = M_{n-ve} \]

\[ A_s = 1 \times 200 = 200 \text{ mm}^2 \]

\[ d = 98 + 214 = 312 \]

\[ a = \frac{A_s f_y}{0.85 f_c' b} = \frac{200 \times 400}{0.85 \times 35 \times 145} = 18.54 \text{ mm} \]

\[ \phi M_{nIII} = 1.0 \times 200 \times 400 \left( 312 - \frac{18.54}{2} \right) = 24.22 \times 10^6 \text{ N.mm} \]

\[ M_w = \phi M_{nl} + \phi M_{nll} + \phi M_{nIII} \]

\[ M_w = 61115 \text{ kN.mm} \]

**Step (2):** \( M_c \) about horizontal axis (Fig. 2)

**Segment I**

Average wall thickness = \((225+285)/2 = 255 \text{ mm}\)

\[ d = 255 - 75 - 8 = 172 \text{ mm} \]

\[ a = \frac{1.0 \times 400}{0.85 \times 35 \times 1} = 13.44 \text{ mm} \]

\[ A_s = 200 \text{ mm}^2 \times 200 \text{ mm} = 1.0 \text{ mm}^2/\text{mm} \]

\[ M_{cl} = 1.0 \times 1.0 \times 400 \left( 172 - \frac{13.44}{2} \right) = 66.311 \text{ kN.mm/mm} \]
Segment II

Assume reinforcement D2 is fully anchored
\[ d = 98+26+16 = 210 \text{ mm} \]

\[ M_{cII} = 1.0 \times (1.0) \times (400) \left( 210 - \frac{13.44}{2} \right) = 81.312 \text{kN.mm/mm} \]

Segment III

\[ d = 214+98+16 = 328 \text{ mm} \]

\[ M_{cIII} = 1.0 \times (1.0) \times (400) \left( 328 - \frac{13.44}{2} \right) = 128.512 \text{N.mm/mm} \]

A weighted average is given by

\[ M_c = \frac{M_{cII} \times 755 + M_{cIII} \times 165 + M_{cw} \times 145}{865} \]

\[ M_c = 79.529 \text{kN.mm/mm} \]

Step (3): Critical wall length \( L_c \) (\( L_4 = 1070 \text{ mm AASHTO} \))

\[ L_c = L + \sqrt{\left( \frac{L}{2} \right)^2 + \frac{8H(M_b+M_w)}{M_c}} \]

\[ L_c = 1070 + \sqrt{\left( \frac{1070}{2} \right)^2 + \frac{8 \times 865 \times (0+61115)}{79.529}} \]

\[ L_c = 2912 \text{ mm} \]

Step (4): Nominal resistance \( R_w \)
\[ R_w = \left( \frac{2}{2L_c - L_t} \right) \left( 8M_b + 8M_w + \frac{M_cL_c}{H} \right) \]

\[ R_w = \left( \frac{2}{2 \times 2912 - 1070} \right) \left( 0 + 8 \times (61115) + \frac{79.529(2912)^2}{865} \right) \]

\[ R_w = 535.6 \text{ KN} > F_t = 240 \text{ KN} \]

**Step (5): Shear transfer between the barrier and Deck**

\[ R_w \text{ spreads out at 1:1 slope from } L_c \]

\[ V_{ci} = \frac{R_w}{L_c + 2H} = \frac{535.600}{2912 + (2 \times 865)} = 115.4 \text{KN/m} \]

Nominal shear resistance \( V_n \) of the interface is given by

\[ V_n = c \cdot A_{cv} + \mu (A_{vf} \cdot f_y + P_c) \]

\[ \leq 0.2 \cdot f_c \cdot A_{cv} \]

\[ \leq 5.5 \cdot A_{cv} \]

\( A_{cv} = \) shear contact area = 410 mm\(^2\)/mm

\( A_{vf} = \) dowel area cross shear plane = 2*1.0 = 2.0 mm\(^2\)/mm

\( f_y = \) yield strength of reinforcement = 400 MPa

\( P_c = \) permanent compressive force = 5.8 N/mm

\( \mu = \) Friction factor = 0.6

\( c = \) cohesion factor = 0.52 MPa

\[ V_n \leq 5.5 \cdot A_{cv} = 5.5 \times 410 = 2255 \text{N/mm} \]
\[ V_n \leq 0.2x(35)x(410) = 2870 \text{N/mm} \]

\[ V_n = 0.52x(410) + 0.6x(2.00x400 + 5.8) \]

\[ V_n = 691.9 \text{ N/mm} \quad > \quad V_{ct} = 115.4 \text{ N/mm} \]

**Minimum dowel**

\[ A_{ct} = 0.35 \frac{b_s}{f_y} = 0.35 \frac{410(200)}{400} = 71.75 \text{mm}^2 < 400 \text{ mm}^2 \text{ (2 bars)} \]

**Minimum development length**

\[ l_{hb} = \frac{100 \cdot d_b}{\sqrt{35}} > 8 \cdot d_b = 8 \times 16 = 128 \text{ mm} \]

\[ > 150 \text{ mm} \]

\[ l_{hb} = \frac{100(16)}{\sqrt{35}} = 270 \text{mm} \]
(1.2) FRP – REINFORCED BARRIERES

\[ \phi = 1.0 \]
\[ f_t = 600 \text{ MPa} \]
\[ f_c = 35 \text{ MPa} \]
\[ A_s (15M) = 198 \text{ mm}^2 \]

**Step (1): \( M_w \) about vertical axis** (Fig. 2)

**Segment I**
\[ A_s = 3 \times 198 = 594 \text{ mm}^2 \]
\[ d = 161 \text{ mm} \]

\[ a = \frac{A_s f_y}{0.85 f_c b} = \frac{594 \times 600}{0.85 \times 35 \times 555} = 21.58 \text{ mm} \]

\[ M_{ai} = 594 \times 600 \left( 161 - \frac{21.58}{2} \right) = 53534 \text{ KN.m} \]

**Segment II**
\[ A_s = 0 \]

**Segment III**
\[ A_s = 198 \text{ mm}^2 \]
\[ d = 312 \text{ mm} \]

\[ a = \frac{A_s f_y}{0.85 f_c b} = \frac{198 \times 600}{0.85 \times 35 \times 145} = 27.54 \text{ mm} \]

\[ \phi M_{ali} = 1.0 \times 198 \times 600 \left( 312 - \frac{27.54}{2} \right) = 35372 \text{ KN.m} \]

\[ M_w = 88905 \text{ KN.mm} \]
Step (2): $M_c$ about horizontal axis (Fig. 2)

**Segment I**

Average wall thickness = $\frac{(225+285)}{2} = 255$ mm  
$d = 255 - 75 - 8 = 172$ mm  
$A_s = 198 \text{ mm}^2 @ 200 \text{ mm} = 0.99 \text{ mm}^2/\text{mm}$  
$a = \frac{0.99 \times 600}{0.85 \times 35} = 19.97 \text{ mm}$

$M_c = 1.0(0.99)(600)(172 - \frac{19.97}{2}) = 96.53 \text{ KN mm/mm}$

**Segment II**

$d = 210$ mm

$M_{cII} = 1.0(0.99)(600)(210 - \frac{19.97}{2}) = 118.809 \text{ KN mm/mm}$

**Segment III**

$d = 328$ mm

$M_{cIII} = 1.0(0.99)(600)(328 - \frac{19.97}{2}) = 188.9 \text{ KN mm/mm}$

$M_c(\text{ave}) = \frac{96.53(555) + 118.809(145) + 188.90(165)}{865}$

$M_c = 116 \text{ KN mm}$

**Step (3): critical wall length $L_c$**

$L_c = \frac{1070}{2} + \sqrt{\left(\frac{1070}{2}\right)^2 + \frac{8 \times 865 \times (0+188.905)}{116}}$

$L_c = 2912 \text{ mm}$
Step (4): Nominal resistance $R_w$

$$R_w = \left( \frac{2}{2L + L} \right) \left( 8M_b + 8M_a + \frac{M_c L_c}{H} \right)$$

$$R_w = \left( \frac{2}{2 \times 2912 - 1070} \right) \left( 0 + 8 \times (88.905) + \frac{120 (2912)^2}{865} \right)$$

$R_w = 782 \text{ KN} > 240 \text{ KN} \quad \text{ok}$

Step (5): Shear transfer between the barrier and Deck

$$V_t = \frac{R_w}{L + 2H} = \frac{781.900}{2912 + (2 \times 865)} = 168.4 \text{ KN/m}$$

$V_n \leq 5.5 \times A_{cv} = 5.5 \times 410 = 2255 \text{ KN/m}$

$V_n \leq 0.2 \times (35) \times (410) = 2870 \text{ KN/m}$

$V_n = C \times A_{cv} + \mu (A_{cf} f_y + P_l)$

$V_n = 0.52 \times (410) + 0.6 \times (1.98 \times 600 + 5.6)$

$V_n = 929.3 \text{ KN/m} \quad > \quad V_{ct} = 168.4 \text{ KN/m}$
(2) PL3 BARRIERS

(2.1) STEEL – REINFORCED BARRIERS

\[ \phi = 1.0 \]
\[ f_s = 400 \text{ MPa} \]
\[ f_c = 35 \text{ MPa} \]
\[ A_s(15M) = 200 \text{ mm}^2 \]
\[ A_s(20M) = 300 \text{ mm}^2 \]

**Step (1):** \( M_w \) about vertical axis (Fig. 3)

**Segment I**
\[ A_s = 5 \times 200 = 1000 \text{ mm}^2 \]
\[ a = \frac{A_s f_y}{0.85 f_c b} = \frac{1000 \times 400}{0.85 \times 35 \times 815} = 16.49 \text{ mm} \]
\[ d_{ave} = 103 + 62 = 165 \text{ mm} \]
\[ \phi M_{nl} = 1.0 \times 1000 \times 400 \left( 165 - \frac{16.49}{2} \right) \]

**Segment II**
\[ A_s = 1 \times 200 = 200 \text{ mm}^2 \]
\[ M_{nl} = 62700 \text{ KN.mm} \]
\[ a = \frac{A_s f_y}{0.85 f_c b} = \frac{200 \times 400}{0.85 \times 35 \times 180} = 14.86 \text{ mm} \]
\[ d = 103 + 164 = 267 \text{ mm} \]
\[ M_{nlII} = 1.0 \times 200 \times 400 \left( 267 - \frac{14.86}{2} \right) \]
\[ M_{nl} = 20766 \text{ KN.mm} \]
Figure A-3. PL3 Barriers
Segment III

\[ A_s = 1 \times 200 = 200 \text{ mm}^2 \]
\[ d = 103 + 229 = 332 \text{ mm} \]

\[ a = \frac{A_s f_y}{0.85 f_d b} = \frac{200 \times 400}{0.85 \times 35 \times 145} = 18.54 \text{ mm} \]

\[ M_{all} = 1.0 \times 200 \times 400 \left( 332 - \frac{14.86}{2} \right) = 25818 \text{ kN mm} \]

\[ M_w = 62700 + 20766 + 25818 = 109284 \text{ kN mm} \]

Step (2): \( M_c \) about horizontal axis (Fig. 3)

Segment I

Average wall thickness = \((225 + 310)/2 = 267.5 \text{ mm}\)
\[ d = 267.5 - 103 = 164.5 \text{ mm} \]
\[ A_s = 300 \text{ mm}^2 @ 200 \text{ mm} = 1.5 \text{ mm}^2/\text{mm} \]

\[ a = \frac{1.5 \times 400}{0.85 \times 35} = 20.17 \text{ mm} \]

\[ M_{cl} = 1.0 (1.5) (400) \left( 164.5 - \frac{20.17}{2} \right) = 92650 \text{ KN mm/mm} \]

Segment II

\[ d = 103 + 164 + 18 = 285 \text{ mm} \]

\[ M_{clt} = 1.0 (1.5) (400) \left( 285 - \frac{20.17}{2} \right) = 164949 \text{ KN mm/mm} \]
Segment III

\[ d = 103 + 229 + 18 = 350 \text{ mm} \]

\[ M_{c_{III}} = 1.0 \times (400 \times \left(350 - \frac{20.17}{2}\right)) = 203.949 \text{ KN/mm} \]

\[ M_{c,II} = \frac{92650(815)+164749(180)+203949(145)}{1140} \]

\[ M_c = 118.190 \text{ KN/mm} \]

Step (3): Critical wall length \( L_c \) \( (L_t = 2440 \text{ mm AASHTO}) \)

\[ L = L_c + \frac{\left(\frac{L_t}{2}\right)^2 + 8H(M_b + M_w)}{M_c} \]

\[ L = \frac{2440}{2} + \sqrt{\left(\frac{2440}{2}\right)^2 + \frac{8 \times 140 \times (0 + 109284)}{118.19}} \]

\[ L_c = 4370 \text{ mm} \]

Step (4): Nominal resistance \( R_w \)

\[ R_w = \left(\frac{2}{2L - L_c}\right)8M_b + 8M_w + \frac{M_cL_c}{H} \]

\[ R_w = \left(\frac{2}{2 \times 4370 - 2440}\right)(0 + 8 \times 109284) + \frac{118.19 \times (4370)^2}{1140} \]

\[ R_w = 906.0 \text{ KN} > 516 \text{ KN} \text{ ok} \]

Step (5): Shear transfer between the barrier and Deck

\[ V_{c,t} = \frac{R_w}{L + 2H} = \frac{906.0}{4370 + (2 \times 1140)} = 136.2 \text{ KN/m} \]
nominal shear resistance $V_n$ of the interface is given by

$$V_n = c A_v + \mu (A_{vf} f_y + P)$$

$$\leq 0.2 f_y A_v$$

$$\leq 5.5 A_v$$

$V_n \leq 5.5 A_{cv} = 5.5 \times 635 = 3492 \text{ N/mm}$

$V_n \leq 0.2 (35) \times 635 = 4445 \text{ N/mm}$

$A_{vf} = \text{dowel area cross shear plane} = 2 \times 1.5 = 3.0 \text{ mm}^2/\text{mm}$

$f_y = \text{yield strength of reinforcement} = 400 \text{ MPa}$

$P_c = \text{permanent compressive force} = 8.02 \text{ N/mm}$

$\mu = \text{friction factor} = 0.6$

$c = \text{cohesion factor} = 0.52 \text{ MPa}$

$$V_n = 0.52 \times (435348) + 0.6 \times (3.0 \times 400 + 8.02)$$

$$V_n = 951.0 \text{ N/mm} > V_{ct} = 136.2 \text{ N/mm}$$

(2.2) FRP – REINFORCED BARRIERS

$\phi = 1.0$

$f_t = 600 \text{ MPa}$
\[ f_c = 35 \text{ MPa} \]
\[ A_s (15M) = 198 \text{ mm}^2 \]
\[ A_s (20M) = 285 \text{ mm}^2 \]

**Step (1):** \( M_{w} \) about vertical axis (Fig. 3)

**Segment I**

\[ A_s = 5 \times 198 = 990 \text{ mm}^2 \]
\[ d_{\text{ave}} = 103 + 62 = 165 \text{ mm} \]

\[ a = \frac{A_s f_y}{0.85 f_c d} = \frac{990 \times 600}{0.85 \times 35 \times 815} = 24.49 \text{ mm} \]

\[ \phi M_{nl} = 1.0 \times 990 \times 600 \left( 165 - \frac{24.49}{2} \right) \]

\[ M_{nl} = 90737 \text{ KN.mm} \]

**Segment II**

\[ A_s = 1 \times 198 = 198 \text{ mm}^2 \]
\[ d = 103 + 164 = 267 \text{ mm} \]

\[ a = \frac{A_s f_y}{0.85 f_c d} = \frac{198 \times 600}{0.85 \times 35 \times 180} = 22.07 \text{ mm} \]

\[ M_{nll} = 1.0 \times 198 \times 600 \left( 267 - \frac{22.07}{2} \right) \]

\[ M_{nll} = 30409 \text{ KN.mm} \]

**Segment III**

\[ A_s = 1 \times 198 = 198 \text{ mm}^2 \]
\[ d = 103 + 229 = 332 \text{ mm} \]
Step (2): $M$, about horizontal axis (Fig. 3)

**Segment I**

Average wall thickness = \((225+310)/2 = 267.5\) mm  
d = 267.5-103 = 164.5 mm  
$A_s = 285\text{ mm}^2 @ 200\text{ mm} = 1.425\text{ mm}^2/\text{mm}$

$$a = \frac{1.425\times 600}{0.85\times 35}=25.11\text{mm}$$

$$M_{cl} = 1.0(1.425)(600)\left(169.5-\frac{25.11}{2}\right)=113.503\text{ KN.mm/mm}$$

**Segment II**

\[d = 103+164+18 = 285\text{ mm}\]

$$M_{cII} = 1.0(1.425)(600)\left(285-\frac{25.11}{2}\right)=203.516\text{ KN.mm/mm}$$

**Segment III**

\[d = 103 + 229 + 18 = 350\text{ mm}\]

$$M_{cIII} = 1.0(1.425)(600)\left(350-\frac{25.11}{2}\right)=252.071\text{ KN.mm/mm}$$
\[ M_c = 145.34 \text{ KN.mm/mm} \]

\[ M_c = \frac{113.503(815) + 203.516(180) + 252.071(145)}{1140} \]

**Step (3): Critical wall length \( L_c \)**  
\((L_t = 2440 \text{ mm AASHTO})\)

\[
L_c = \frac{L_t}{2} + \left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_w)}{M_c}
\]

\[
L_c = 2440 + \left(\frac{2440}{2}\right)^2 + \frac{8 \times 140 \times (0 + 158952)}{145.34}
\]

\[ L_c = 4605 \text{ mm} \]

**Step (4): Nominal resistance \( R_w \)**

\[
R_w = \left(\frac{2}{2L_c - L_t}\right) \left(8M_b + 8M_w + \frac{M_c L_c}{H}\right)
\]

\[
R_w = \left(\frac{2}{2 \times 4605 - 2440}\right) \left(0 + 8 \times 158952 + \frac{145.34 \times (4605)^2}{1140}\right)
\]

\[ R_w = 1175.0 \text{ KN} > 516 \text{ KN} \quad \text{ok} \]

**Step (5): Shear transfer between the barrier and Deck**

\[
V_{ci} = \frac{R_w}{L_t + 2H} = \frac{1175000}{4605 + (2 \times 1140)} = 170.6 \text{ KN/m}
\]

nominal shear resistance \( V_n \) of the interface is given by

\[ V_n = cA_v + \mu(A_v f_v + P) \]
\[ \leq 0.2f_cA_v \]
\[ \leq 5.5A_v \]

\[ V_n \leq 5.5A_{cv} = 5.5 \times (635) = 3492 \text{ N/mm} \]

\[ V_n \leq 0.2x(35)x(635) = 4445 \text{ N/mm} \]

\[ A_{vf} = \text{dowel area cross shear plane} = 2 \times 1.425 = 2.85 \text{ mm}^2/\text{mm} \]
\[ f_y = \text{yield strength of reinforcement} = 600 \text{ MPa} \]
\[ P_c = \text{permanent compressive force} = 8.02 \text{ N/mm} \]
\[ \mu = \text{friction factor} = 0.6 \]
\[ c = \text{cohesion factor} = 0.52 \text{ MPa} \]

\[ V_n = 0.52 \times (435) + 0.6 \times (2.85 \times 600 + 8.02) \]

\[ V_n = 1257.0 \text{ N/mm} \quad > \quad V_{ct} = 170.6 \text{ N/mm} \]