ENHANCING DUCTILITY OF WWR SLABS

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ABSTRACT

A series of research studies have recently identified an issue called strain localization in welded wire reinforced (WWR) members. This phenomenon reportedly concentrates strains at welded cross wire locations and severely limit ductility. Those that identified the phenomenon used it to imply that WWR is unsafe because it does not warn of failure. This research program is investigating details to mitigate the strain localization effect and demonstrate the WWR can be used safely. Sixteen beams have been constructed using WWR and rebar with various cross wire spacing, using a realistic design. The strain localization phenomenon was not demonstrated, but WWR slabs are somewhat less ductile than traditionally reinforced members. The WWR members were shown to provide adequate ductility for warning of impending failure visually and with a well-accepted ductility measure. Future study will focus on proving ability of WWR to provide load redistribution, investigating the effect of cross wire diameter on strain localization and developing simple and easy to use guidelines for proper WWR detailing.

Keywords: Welded Wire Reinforcement, One-Way Sabs, Ductility, Strain Localization.

INTRODUCTION

There are many advantages to using welded wire reinforcement (WWR) in reinforced concrete rather than conventional rebar. WWR has a higher yield strength (usable up to 80 ksi), reducing the amount of needed steel by 33%. Time and labor required to place WWR is significantly less than rebar and has an inherently higher accuracy for reinforcement spacing due to the mechanized welding process.

Gilbert and Smith[1] identified a phenomenon termed strain localization, which significantly reduces member ductility, raising concern over visible warning signs and moment redistribution. The strain localization phenomenon occurs as the concrete cracks near cross weld locations and concentrates strain over a small length of the longitudinal wire due to the great bond created by the wire deformations and cross welds. This results in much less accumulated curvature and global deformation. Gilbert and Smith[1] tested eight total WWR reinforced slabs with relatively small spans, using 8 in. cross wire spacing. Their program, results presented in Table 1, had beams below the ACI 10.5.1[2] beam minimum reinforcement ratio and was very near the ACI 7.12.2.1[2] temperature and shrinkage minimum. Interestingly, there was a clear trend of higher reinforcing ratios resulting in higher deflections at failure. From Gilbert and Smith[1] it is clear that strain localization exists, but the specimens were so close to the minimums (which are intended to maintain ductility post-cracking) that this appeared to exacerbating the phenomenon.

Table 1 – Gilbert and Smith[1] Experimental Results Summary (Adapted)

Specimen	Supports	fy, psi	Specimen ρ	ρmin per 7.12.2.1	ρmin per 10.5	Maximum Deflection (in.)
1	SS†	93000	0.0018	0.014	0.260	1.71
2	SS	85000	0.0029	0.014	0.282	2.05
3	SS	81000	0.0046	0.014	0.292	3.31
4	CΨ	93000	0.0018	0.014	0.260	-
5	C	93000	0.0018	0.014	0.260	0.45
6	C	93000	0.0018	0.014	0.260	0.28
7	SS	85000	0.0016	0.014	0.437	0.57
8*	SS	76000	0.0038	0.014	0.307	8.27

*control sample reinforced with hot rolled rebar
† simply supported one-way slab
ΨContinuous one-way slab

Building on this work, Tuladhar and Lancini[3] attempted to address the low ductility by including steel fibers in the concrete mixture, with the intent of better distributing cracks more and therefore, more uniformly distributing rebar strains. This research program had identical cross wire spacing to the Gilbert and Smith[1] program, but smaller spans and higher reinforcing ratios. Test details and results for the Tuladhar and Lancini[3] study are presented in Table 2. It seems a similar phenomenon of strain localization affected the maximum deflections in this experiment as well (compare the control Specimen 1 peak load deflection of 2.17 in. to Specimen 2-8 peak load deflections between 0.51 in. through 0.87 in. in Table 2). However, comparing the no fiber Specimen 2 deflection (see 0.51 in. in Table 2) to the best WWR reinforced specimen, (see Specimen 6 at 0.87 in. in Table 2), results in a significant 71% relative increase, but a marginal absolute 0.36 in. increase. Such an increase is probably within the error of a full-scale test. Interestingly, the maximum observed deflections for the fiber and non-fiber reinforced WWR specimens resulted in nearly identical maximum deflections which might be considered more important for redistribution of moments in an overload scenario.

Table 2 – Tuladhar and Lancini[3] Experimental Results Summary (Adapted)

Specimen	Steel	d_b (in.)	Ast (in.^{2})	ρ (%)	Fiber Type	Yield Deflection (in.)	Peak Load Deflection (in.)	Maximum Deflection (in.)
1	Rebar	0.47	0.70	0.67	None	0.55	2.17	3.54
2	Wire	0.30	0.55	0.52	None	0.39	0.51	1.30
3	Wire	0.30	0.55	0.52	Crimped	0.43	0.67	1.42
4	Wire	0.30	0.55	0.52	Crimped	0.43	0.59	1.34
5	Wire	0.30	0.55	0.52	Hooked	0.55	0.75	1.30
6	Wire	0.30	0.55	0.52	Hooked	0.47	0.87	1.38
7	Wire	0.30	0.55	0.52	Twincone	0.43	0.83	1.38
8	Wire	0.30	0.55	0.52	Twincone	0.39	0.79	1.38

While somewhat successful, and using higher reinforcing ratios than Gilbert and Smith[1], the steel fibers would not likely be a viable construction option in the current US market due to costs.

Sakka[4] tested four simply supported one-way slabs and seven continuous slabs, the simply supported slabs were 2.8 ft in width, 3.9 inches in depth and 8.2 ft in length with very small cross-weld spacing of 7.87 in., with applying only a single point load at the mid span, the continuous slabs dimensions and cross-weld spacing was similar as simply-supported slabs but the length were 14 ft. The maximum deflection for all tested slabs reinforced with WWR ranged between 0.78 – 1.77 in. and the failure modes were sudden collapse and fracture of reinforcement without crushing of the compressive concrete on the critical section, while slabs reinforced with conventional rebar reported higher deflection 3.74 in. to 7.1 in., and failed by crushing concrete first.

The strain localization phenomenon has not been noticed in members like beams with WWR as longitudinal or shear reinforcing[5,6,7]. It seems that the strain localization is limited to thin or lightly reinforced members like slabs. To address the above issues and start to understand the strain localization phenomenon, the experimental program in this paper includes serviceability considerations like deflection and crack control to ensure realistic longitudinal and transverse reinforcing spacing and realistic span-to-depth ratio. This research will experimentally compare strength and ductility of traditionally reinforced concrete slabs to WWR reinforced concrete slabs.

EXPERIMENTAL PROGRAM

SLAB SPECIMENS

In this paper, load tests were carried out on 16 simple span, one-way concrete slabs (see Table 3). A prototype structure was designed assuming gravity loading of 10 psf super imposed dead load and 50 psf live load for a 20 ft span with a common span to depth ratio of 34 (7 in. cross section depth). Concrete strength was specified to be 5000 psi. Longitudinal reinforcing was designed to meet flexural strength requirements for design-level loading using strain compatibility and the Whitney stress block, ignoring any potential compression reinforcement and using 0.75 in. clear cover. Service deflections (including long term creep/shrinkage per ACI Eq. 9-11[2]) were limited to total deflection of 0.75 in. and checked using an average effective moment of inertia (ACI Eq. 9-8[2]). Crack widths were checked using the method developed in Frosch (1999) and checked against 0.018 in to maintain reinforcement size and spacing as would be common in design. Shear was checked and found not to require reinforcing as is typical in slab structures. The welded wire design is assumed to be a one-to-one replacement of all rebar and the design should remain the same (i.e. the larger yield strength of the WWR was ignored, as is common).

The test specimens were developed to match the rebar size, spacing and reinforcing ratio of the prototype structure. A representative slab width of 24in. was selected to remain economical, while still conforming to the above prototype design. See Figure 1 for cross section dimensions. Cross wires were provided as tied loose wires at 14 in. spacing, however cross welds were provided for slabs with D20 wires at spacing as presented in Table 3. The loose wires (i.e., non-welded) were tied like traditional rebar. Specimens SW17 and SW18 were all loose wire and contained no welded cross wires so that there could be a direct comparison without any possible strain localization. For the Grade 60 rebar slabs, SR3 and SR4, traditional rebar provided orthogonal bars at 14 in. spacing for comparison purposes.

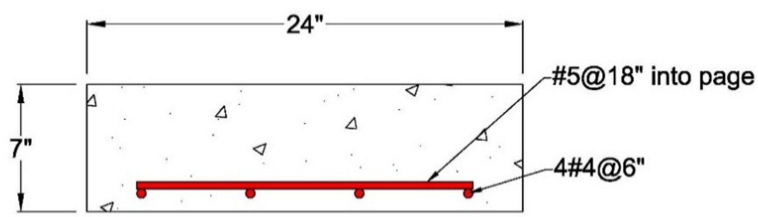

Figure 1 – Simple Span Specimen Cross Section

Table 3 – Slab Specimen Reinforcing Steel and Crosswire Information

Specimen	Reinforcing Steel	Equal Weld Spacing (in.)
SR3	Rebar	-
SR4	Rebar	-
SW5	WWR	80
SW6	WWR	80
SW7	WWR	60
SW8	WWR	60
SW9	WWR	48
SW10	WWR	48
SW11	WWR	40
SW12	WWR	40
SW13	WWR	35
SW14	WWR	35
SW15	WWR	30
SW16	WWR	30
SW17	WWR	-
SW18	WWR	-

TEST SETUP

Figure 2 presents a drawing of the test setup and Figure 3 presents a photo. Four point loads were applied using spreader beams to closely mimic a distributed load and the applied load was measured at the ram location using an electrical resistance strain gage based full bridge load cell. Deflections were monitored at third points along the slab using wire potentiometers.

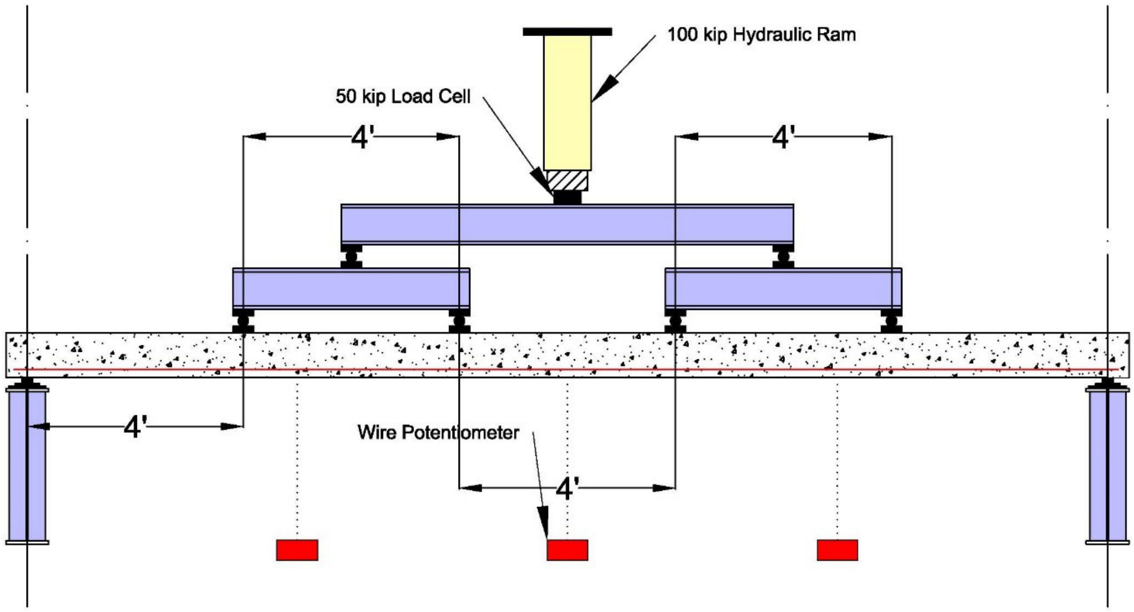

Figure 2 – Simple Span Specimen Test Setup and Instrumentation (not to scale, symmetric about centerline)

Figure 3 – Photo of 20 ft Simple Span Slab Test Setup

MATERIAL TESTS

The concrete was ordered from a local ready-mix company, the wire was provided by an east coast wire producer and a local construction supply company provided the rebar. Several 4 in. x 8 in. cylinders were obtained from the middle of each concrete truck batch for each concrete pour. Compressive strength, split tension strength and elastic modulus were obtained per ASTM C39, ASTM C496 and ASTM 469, respectively on the day of each slab test. Tensile testing of the reinforcing steels followed ASTM A370 and full stress versus strain curves were obtained as well as yield stress, ultimate stress and ultimate elongation.

EXPERIMENTAL RESULTS

MATERIAL TESTING RESULTS

Concrete cylinders were tested each day of slab testing with mechanical properties reported in Table 4. The concrete slabs were cast at two separate times and slabs were tested as close to the intended 5,000 psi as possible. For some reason, the 9/11/2015 concrete did not reach strength, but specimens were tested anyway due to lab time and space requirements. Reported results are the average of three cylinders for compressive strength (f'_c), three for split tension strength (f_r) and three for modulus of elasticity (E_c).

Table 4 – Concrete Mechanical Properties

Slabs	f'_c (psi)	f_r (psi)	E_c (ksi)	Pour Date	Test Date
SR3	5,739	312	3,420	8/21/2015	9/16/2015
SR4	3,591	340	3,150	9/11/2015	10/12/2015
SW5	6,066	321	3,540	8/21/2015	9/21/2015
SW6	3,591	340	3,148	9/11/2015	10/12/2015
SW7	6,197	324	3,555	8/21/2015	9/23/2015
SW8	3,704	348	3,244	9/11/2015	10/14/2015
SW9	6,197	324	3,555	8/21/2015	9/23/2015
SW10	3,949	369	3,483	9/11/2015	10/19/2015
SW11	6,328	331	3,638	8/21/2015	9/25/2015
SW12	3,949	369	3,483	9/11/2015	10/19/2015
SW13	5,870	319	3,500	8/21/2015	9/18/2015
SW14	4,097	377	3,578	9/11/2015	10/21/2015
SW15	5,608	309	3,341	8/21/201	9/14/201

				5	5
SW16	4,097	377	3,578	9/11/2015	10/21/2015
SW17	5,608	309	3,341	8/21/2015	9/14/2015
SW18	3,423	328	3,004	9/11/2015	10/9/2015

STEEL TESTING RESULTS

Three samples for each reinforcing steel heat were tested and stress versus strain curves developed using an 8 in. extensometer as presented in Figure 4. The differences in steel ductility are very clear from the stress strain diagrams and the average elongation presented in Table 5. Table 5 also presents the average yield and average ultimate strengths for the bars. Average yield for both the rebar and WWR were considerably higher than used in a typical design (i.e., f_y = 60 ksi and f_y = 80 ksi, respectively).

Figure 4 – Stress versus Strain Rebar and WWR used in SR3-SR18

Table 5 – Average Steel Properties

Specimen	Steel Type	Average Yield Stress (ksi)	Average Ultimate Stress (ksi)	Elongation (%)
SR3,SR4	Rebar	78.6	121.3	13.1
SW5 - SW18	WWR	104.6	117.4	4.7

SLAB TESTING

Sixteen concrete one-way slabs were tested with rebar (SR3 and SR4) and WWR (SW5 through SW18). Failure modes were markedly different. Figure 5 through Figure 7 present photos of the typical failure modes for the Grade 60 rebar and the Grade 80 WWR slabs. The Grade 60 rebar presented very high ductility and in both cases touched the ground without a true failure (see Figure 5). Additional deflection could have been applied to SR3 and SR4; however, the support height prevented additional deformation (24 in.). The WWR slabs did display significant deflection, as shown in Figure 6a for SW15 and Figure 6b for SW16, that would be easily identified by an occupant as impending failure. However, the slabs with WWR did typically fail more suddenly as the WWR reached its maximum strain and ruptured (see SW14 in Figure 7).

Figure 5 – Maximum deflection of SR3, note the well distributed cracking

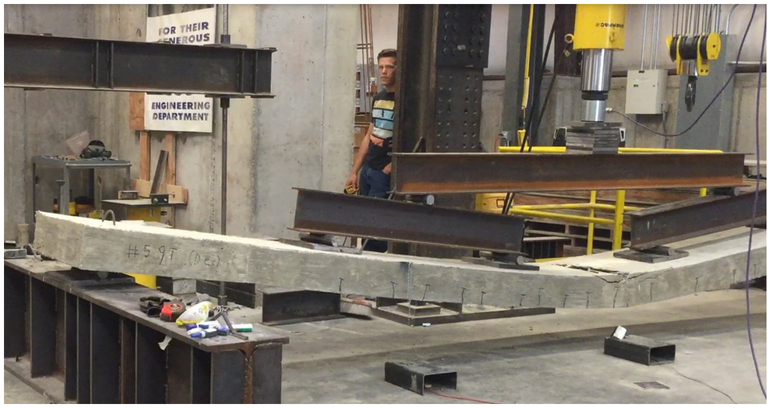

Figure 6 – Maximum deflection of (a) SW15 just prior to failure (b) SW16 deflection at failure (note the compression failure in concrete at mid-span)

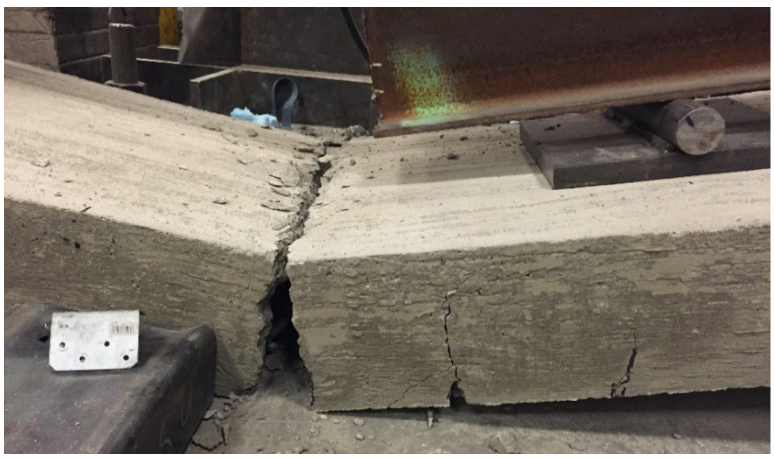

Figure 7 – SW14 Ruptured Wire Failure

Slabs exhibited the typical load versus deformation behaviors presented in Figure 8. The Grade 60 rebar slabs deflected until they hit the floor gradually, although they were clearly past maximum strength. For instance, Figure 8 shows SR3 peak load occurred at approximately 16 in. of deflection with a slight drop in load, but a high level of load maintained through the remaining deformation. In contrast, SW13 and SW6, for instance, hit peak load at significantly less than the rebar slab and maintained just lower than maximum until WWR rupture for only a small amount of plastic deformation. All WWR slabs exhibit similar behavior with relatively small post-peak deformation capability and the Grade 60 rebar slabs exhibit very good post-peak ductility. In addition, because of the difference in steel strengths, there was a clear increase in strength for the WWR slabs over the Grade 60 slabs. Table 6 presents the maximum load, yielding deflection, deflection at peak load, maximum attained deflection and ductility (discussed in the next section). Unfortunately, the data acquisition system recorded a corrupted file for SR4 and a Load versus Deflection plot could not be constructed. However, for all tests the maximum load and maximum deflection were manually recorded to mitigate such an equipment failure and these values are reported in Table 6 for SR4.

It is clear from the Table 6 deflections that there is no significant trend between crosswire location, load or deflection. SW17 and SW18 exhibited some of the lowest deflection measurements and used only loose wire without any welded cross wires. The other WWR reinforced slabs were able to match or improve upon SW17 and SW18 deflection and ductility. This finding indicates that the strain localization phenomenon is not present in the tested slabs. The slabs tested in this study contain larger cross wire spacing than the other studies discussed above, as well as higher reinforcing ratios similar to a slab or wall that would be present in a building.

Figure 8 – Midspan Deflection for representative slabs SR3, SW6 and SW13

Table 6 – Maximum Load, Important Deflections and Ductility

Slabs	Maximum Load (psf)	Max Deflection @ Midspan (in.)	Deflection at yielding Δy (in.)	Deflection at Peak Load Δu (in.)	Ductility Ratio, μ_Δ
SR3	217	23.3*	2.3	16.3	7.08*
SR4†	191†	24.5†	-	-	-
SW5	214	13.9	3.5	12.7	3.6
SW6	291	13.1	3.6	9.3	2.6
SW7	225	13.1	4.0	12.2	3.1
SW8	247	11.5	4.0	9.8	2.5
SW9	261	12.9	4.5	14.5	3.2
SW10	287	12.5	3.8	9.5	2.5
SW11	250	12.3	3.8	11.4	3.0
SW12	255	10.9	4.2	10.5	2.5
SW13	231	13.5	3.9	12.4	3.2
SW14	279	13.3	3.9	9.6	2.5
SW15	262	11.1	3.5	10.0	2.9
SW16	236	11.0	4.2	10.1	2.4
SW17	272	12.3	3.8	9.8	2.6
SW18	276	10.1	4.3	9.4	2.2

*Underestimated due to slab hitting floor
†Data file became corrupted, hand recorded value

DISCUSSION ON DUCTILITY

Because of the clear differences in ductility of the steel materials themselves, it is expected that the member ductility would be significantly different for the WWR and Grade

60 reinforced slabs. However, the presence of strain localization is either minimized by the design or the as tested crosswire spacing. Regardless, the ACI 318[2] concrete code does not have guidance on – or an accepted measure of – the amount of ductility a concrete member must have for a certain system (i.e., gravity versus lateral force resisting system). However, the Australian Concrete Code AS3600[8] does have a recommendation based on the ductility ratio:

(1)

Where μ_Δ is the ductility ratio, Δ_u is the deflection at peak load and Δ_y is the deflection at yield. Figure 9 presents a visual determination of these variables. Table 6 presents ductility ratios for the rebar and wire slabs. The Grade 60 rebar slabs obtained a ductility ratio of approximately 7, but this is probably underestimated due to the deflection limitation of the test setup. The WWR slabs all obtained ductility ratios between 2.2 and 3.6.

Figure 9 – Example Calculation of Δ_y and Δ_u (on SW14)

AS3600[8] allows the use of members with ductility ratio in excess of 2.0 for gravity load situations, but does not allow for moment redistribution design, which is uncommon. It should be noted that both Sakka[4] and Tuladhar and Lancini[3] used this same measure and the slabs contained in the current research exhibited significantly better deflections and ductility. This is likely due to the more realistic designs and cross wire spacing investigated.

Because the literature found strain localization with 8 in. cross weld spacing and the testing presented herein investigated cross weld spacing down to 30 in., it is recommended that cross weld spacing be limited to a minimum of 30 in. for high ductility members like slabs. With additional testing of closer cross welds, this could be decreased.

If necessary, due to minimum spacing requirements or strength, additional cross wires (structural or temperature and shrinkage) can be provided by a nested WWR mat or welds can be skipped (possible with some manufacturers, but likely a significant request), provided that the 30 in. spacing is maintained along any individual wire. These solutions would not appreciably affect installation times, WWR's most time and cost saving quality.

CONCLUSIONS

The strain localization effect was recently identified as a potential safety concern from previous researchers. The above paper has presented the results from 16 realistically designed one-way slab tests using traditional Grade 60 rebar and Grade 80 WWR to identify mitigating strategies for strain localization. Crosswire spacing was varied on the Grade 80 WWR slabs to investigate the ductility reducing effects of strain localization. The following conclusions can be made from the experimental program:

1. As expected from lower steel ductility WWR mat reinforced slabs result in higher strength and lower ductility when rebar is replaced with the same area of WWR.

2. Using slabs detailed similar (reinforcing ratio, span to depth ratio, bar diameter) to those designed in practice, the strain localization phenomenon was not observed. Based on this testing, designers are recommended to maintain at least 30 in. cross wire spacing. To conform to ACI 318-14 Section 7.7.6 (maximum spacing of shrinkage and temperature reinforcement perpendicular to the flexural reinforcement as lesser of 5h and 18 in.) it is recommended to provide additional cross wires by nesting mats, but keep cross weld spacing on any given wire above 30 in., or skip welds. The latter option should be possible with modification to modern WWR machines but will be a special request.

3. Based on the limited study presented here, slabs reinforced with WWR can provide adequate ductility for gravity systems based on the measures outlined in the Australian Concrete Structures Standard AS3600[8]. It is recommended that ACI also develop a similar recommendation for adequate ductility in gravity load situations.

FUTURE RESEARCH

This research in ongoing and future experiments will try to identify effects of strain localization in order to develop mitigation strategies. Different wire sizes, cross wire spacing and specimen configuration will all be investigated. Understanding the phenomenon of strain localization is paramount to maintaining member ductility when designing high ductility members like slabs with WWR. Future design and detailing recommendations will be developed.

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