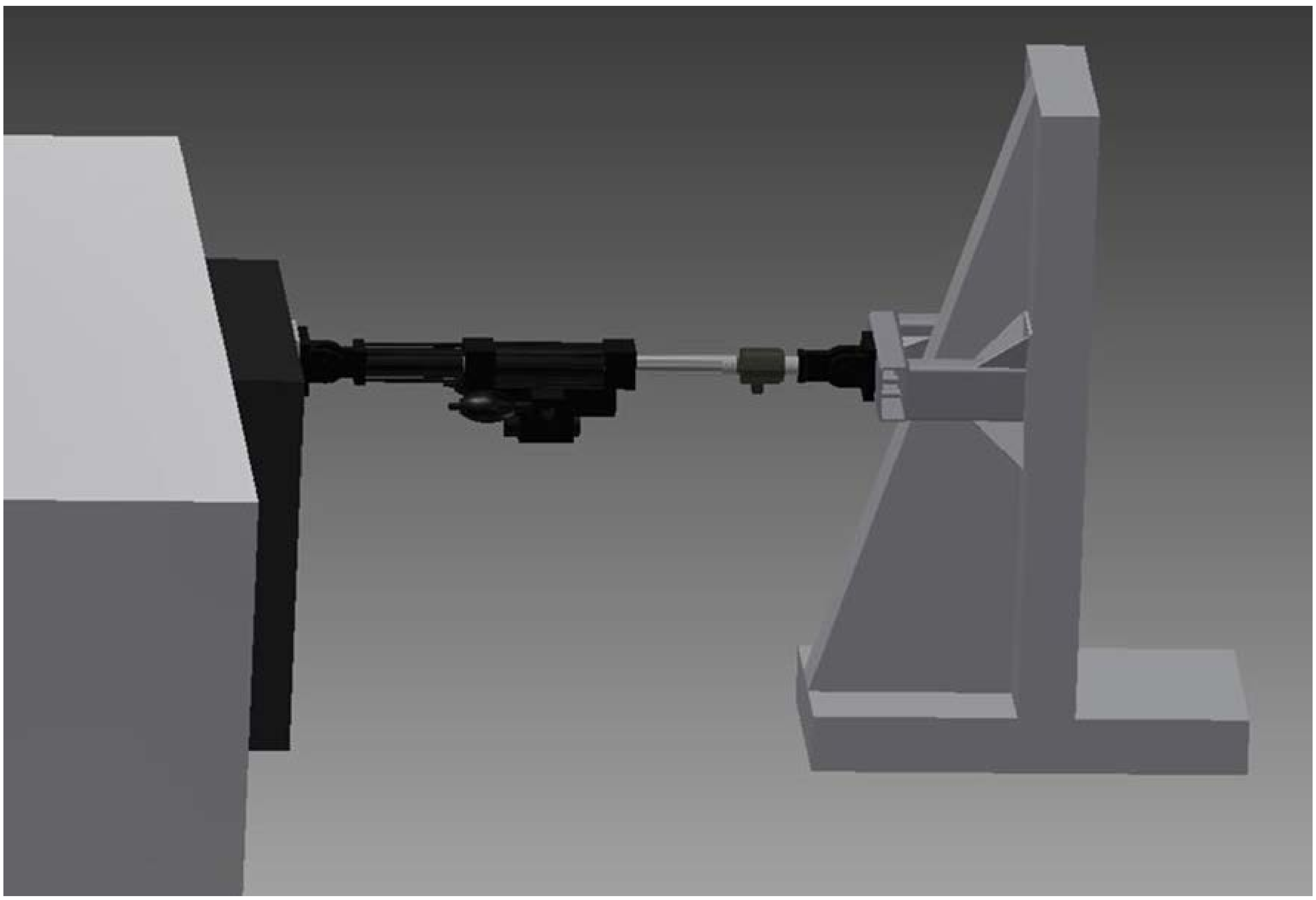
RECLAMATION

Managing Water in the West

Report DSO-2015-01

Spillway Wall Structural Capacity

Dam Safety Technology Development Program



U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

December 2014

1. REPORT DATE
December 2014

2. REPORT TYPE
Final

3. DATES COVERED ***(From - To)***

4. TITLE AND SUBTITLE
Spillway Wall Structural Capacity

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
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5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
Bureau of Reclamation, Denver Federal Center,
P.O. Box 25007, Denver, CO, 80225

8. PERFORMING ORGANIZATION REPORT NUMBER

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Bureau of Reclamation, Denver Federal Center,
P.O. Box 25007, Denver, CO, 80225

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT

13. SUPPLEMENTARY NOTE

14. ABSTRACT

This report compares how well finite element analyses of spillway counterfored walls match lab tests of a model of a typical counterforted wall. This is part of an ongoing validation study to gain confidence in using finite element models to predict the seismic performance of dams.

15. SUBJECT TERMS
Reclamation, spillway wall, concrete material models, finite element analysis

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	a. THIS PAGE			Roman M. Koltuniuk
U	U	U		27	19b. TELEPHONE NUMBER *(Include area code)* 303-445-3225

Standard Form 298 (Rev. 8/98)
Prescribed by ANSI Std. Z39.18

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Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Bureau of Reclamation

Dam Safety Technology Development Program
Waterways and Concrete Dams Group 1, 86-688110

DSO-2015-01

Spillway Wall Structural Capacity

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REVISIONS					
Date	Description	Prepared	Checked	Technical approval	Peer review

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Acronyms and Abbreviations

CSCM	Continuous Surface Cap Model
KCC	Karagozian Case Concrete
psi	pounds per square inch
EOS	Equation of State
MERL	Materials and Engineering Research Laboratory

I. Introduction

Reclamation has numerous spillways and other earth retaining structures founded on soil or weathered rock and backfilled with various embankment materials, some in high seismic areas. Many of these spillways have counterforted spillway walls. Finite element analyses of these walls performed by Reclamation are being used more often to predict wall behavior from seismic lateral earth pressures. Recent finite element analyses of the Bradbury and Scoggins dams included a full 3-D representation of these spillways including concrete material models that predict cracking and crushing along with nonlinear reinforcement. Results of these high level analyses indicate that the counterforted spillway walls can sustain greater seismic loads than would otherwise be calculated using Code design. This is of no surprise, since design by Code procedures should yield a conservative result.

There are multiple challenges in estimating the performance of spillway walls during a seismic event. The first is predicting the reserve capacity in a wall. Typically these walls lack seismic reinforcement detailing. Seismic reinforcement layout details are something the finite element codes cannot incorporate easily. A second challenge is introducted in evaluating seismic soil loads and how spillway walls behave at full scale. A full scale, coupled analysis is unsurpassed in modeling the true soil structure interaction, however, the cost can be prohibitive due to the size of such a model.

This research will concentrate on determining the structural capacity of a scaled counterforted spillway wall. However, instead of placing soil behind the wall and loading the wall with this backfill, a loading ram will be used in the laboratory. This simplification will still give a good indication of the actual capacity of these walls but without the complicated behavior of soil loading, which introduces an added uncertainty to the system. Finite element analyses of this wall (using the LS-DYNA [1] Finite Element Software) will also be done as part of an ongoing effort to validate the use of these finite element codes.

II. Lab Model

This section will summarize the design, details and testing of the scale model of the counterfort wall in Reclamation's Materials and Engineering Research Laboratory (MERL).

A. Design of Model

It is important to pick the correct size of the lab model such that one can capture all pertinent desired effects. These include choosing a model size big enough that enough rebar detail can be properly represented, yet small enough that construction can be performed in a lab environment. Upon consultation with the labs, it was determined that a nine foot high wall would be constructed. Both wall panels and counterfort thicknesses were designed as eight inches so that two layers of rebar could be inserted. The heel and toe were also designed to be eight inches thick. The sinusoidal loading was applied at the mid-height of the wall. This facilitates matching the bolt pattern on the labs strong wall.

B. Model Details

Figure 1 shows the dimensions of the counterfort wall as constructed. Wall proportioning is similar to an actual scaled counterforted wall, although it is recognized that some spillways are constructed much thinner than these proportions.

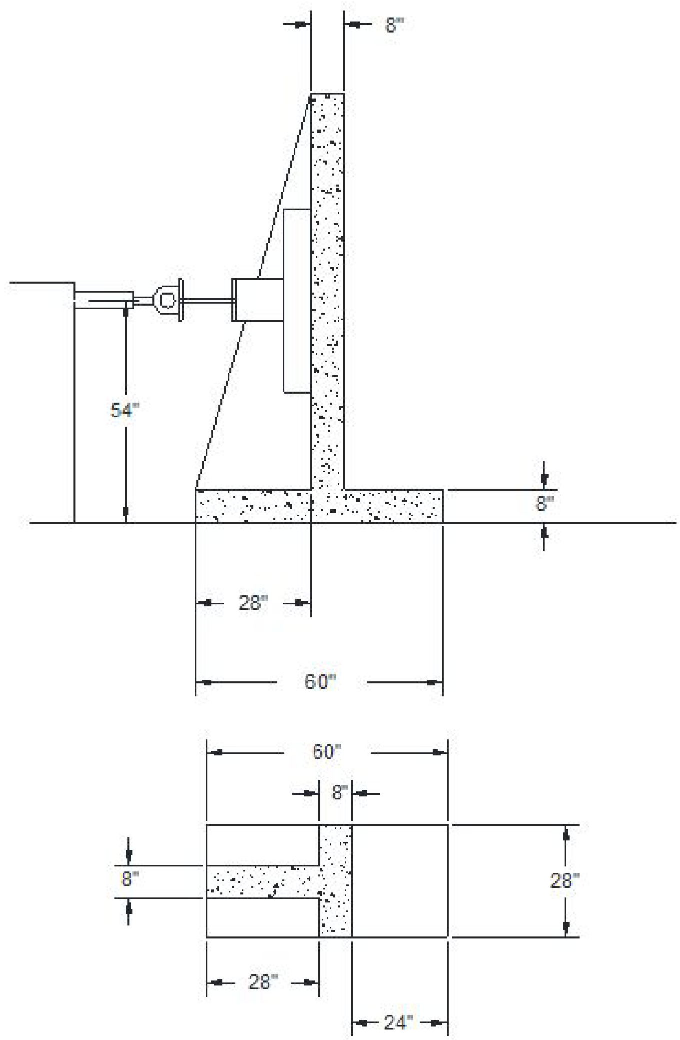


Figure 1. – Counterfort layout.

An initial reinforcement design was done to determine the size of the steel required. Finite element analysis using this pattern was run in order to determine the behavior of the counterfort wall. The final steel pattern used was based on a modified sizing of the counterfort steel in order to facilitate a more ductile response. The wall had to show significant damage while staying within the capacity of the 22 Kip ram available in the labs. Minimum shear steel was used in order not to contribute to the capacity of the wall. Figure 2 shows the final reinforcement layout used in the construction of the scaled wall. All reinforcement are #3 bars, with an assumed f_y = 60,000 pounds per square inch (psi).

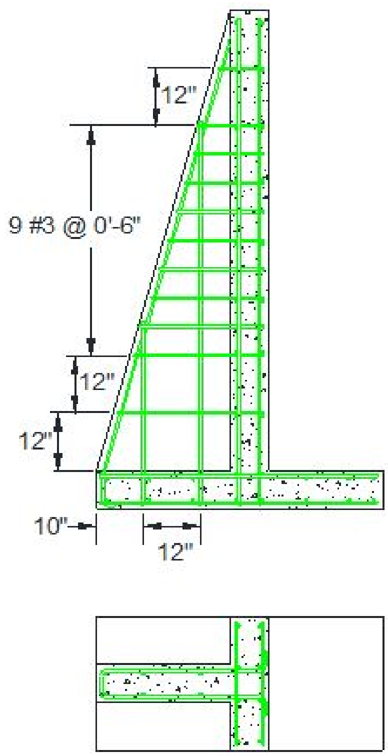
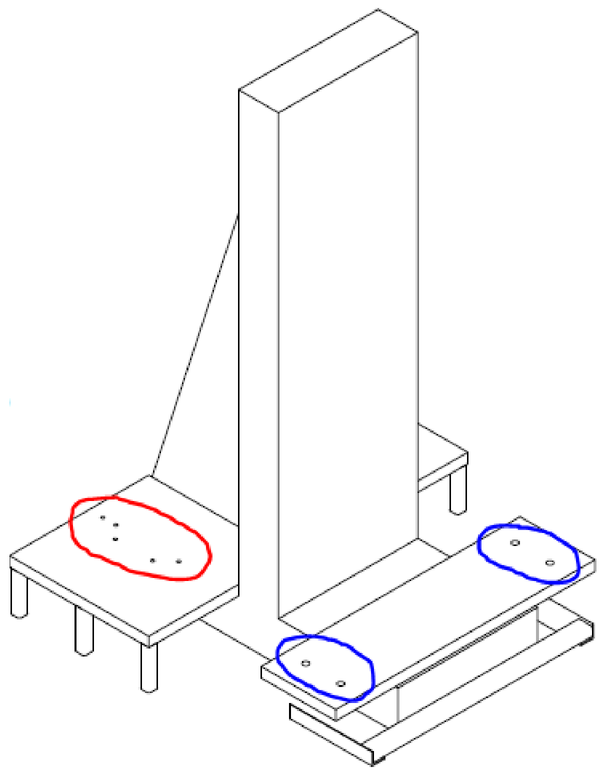


Figure 2. – Final reinforcement layout.

The concrete design was modified to produce a lower strength concrete with a f'_c = 2,500 psi using a maximum aggregate size of ¾ inch.

C. Model Testing

The wall dimensions and final rebar layout was given to the labs for construction. The labs had to design a hold down system in order to keep the wall from moving during the loading, but in keeping with existing bolt hole patterns available on the lab floor. Figure 3 and Figure 4 show this hold down system.



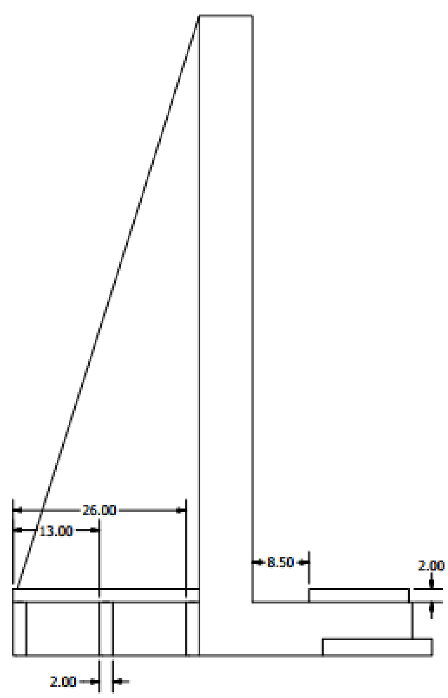


Figure 3. – Details of hold down.

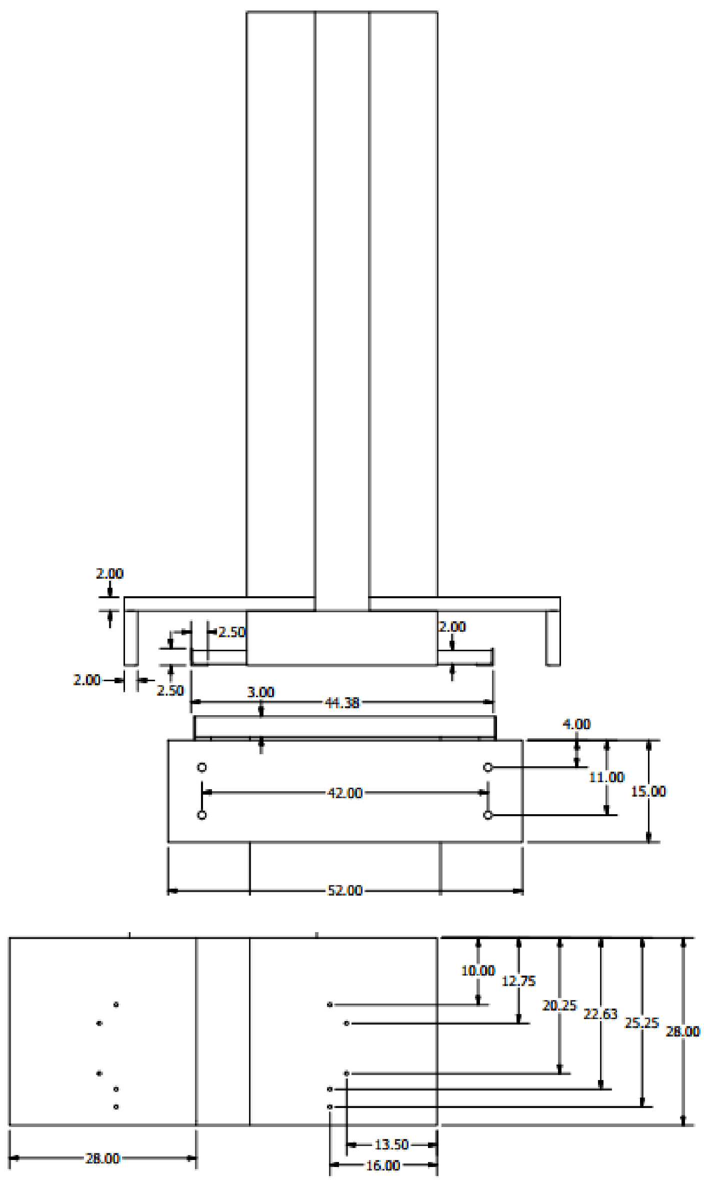


Figure 4. – Details of hold down (continued).

The bolts holding down the heel (shown in red in Figure 3) are ½ inch diameter with a tensile strength of 125,000 psi. The bolts holding down the toe (shown in blue Figure 3) are 1 inch diameter with a tensile strength of 110,000 psi.

The initial loading provided to labs is shown in Figure 5. This is a displacement control loading and shows a gradual increase in wall displacement followed by a series of low amplitude cycles in order to represent a seismic load. These loadings were allowed to be adjusted as appropriate during the experiment.

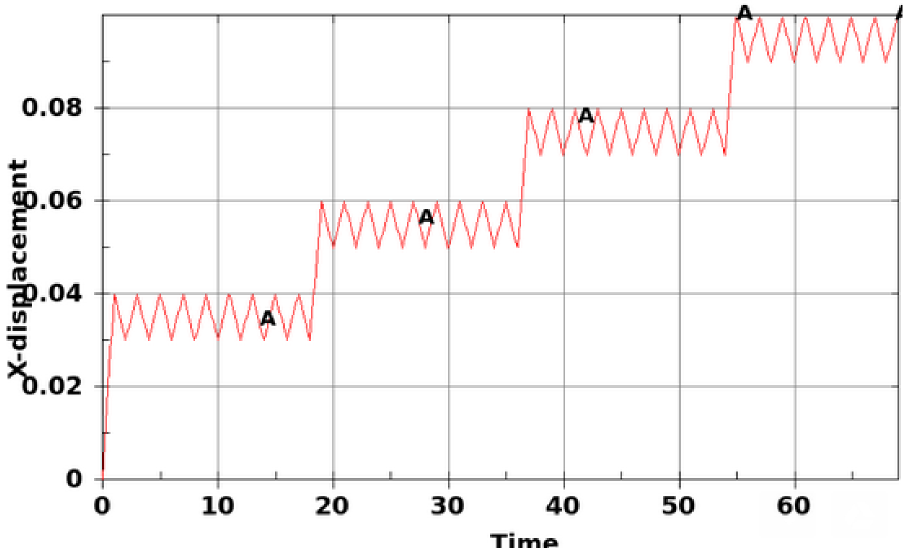


Figure 5. – Initial loading.

Material testing of the reinforcement showed that the actual yield of the #3 bars, was f_y = 79,222 psi (average of 2 tensile tests). The concrete exhibited an f'_c = 2,810 psi at the time of the test.

D. Results

The lab results are presented in a series of tests. Each test consists of a displacement of the wall followed by 10 cycles of loading simulating an earthquake type motion. Figure 6 shows the readings for one of these tests early in the experiment.

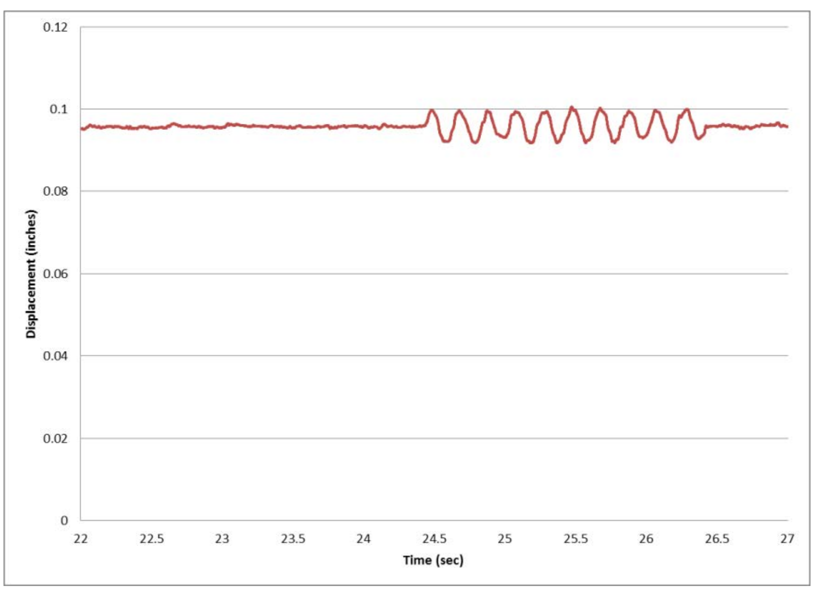


Figure 6. – One of the early cyclic loadings.

The following test would start out with the final displacement of the previous test. Figure 7 shows the cyclic loading later in the experiment. Note that the magnitude of the displacement increases as the test progresses.

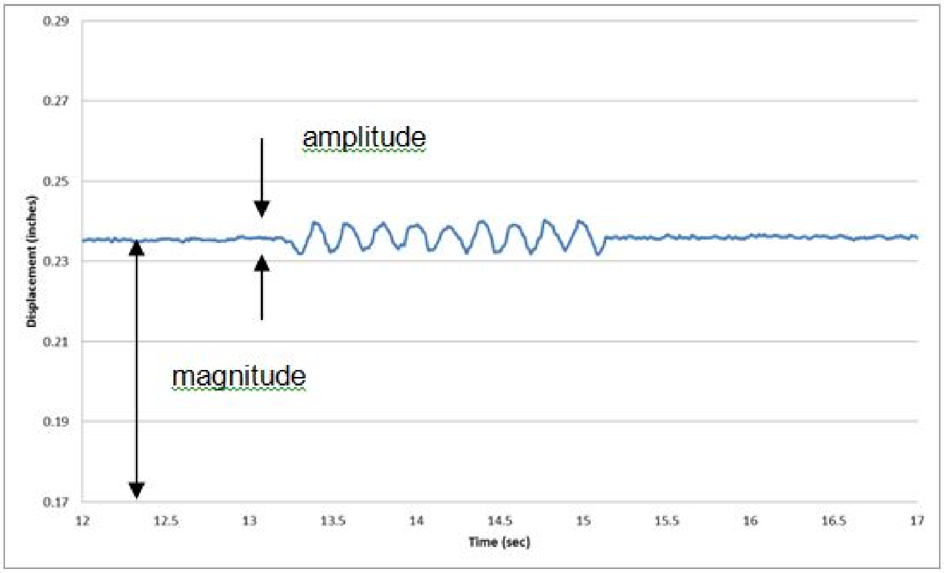


Figure 7. – Cyclic loading.

As the experiment progressed, the amplitude of the cycles was also increased in order to increase damage to the counterforted wall. Figure 8 and **Figure 9** show this increase in amplitude.

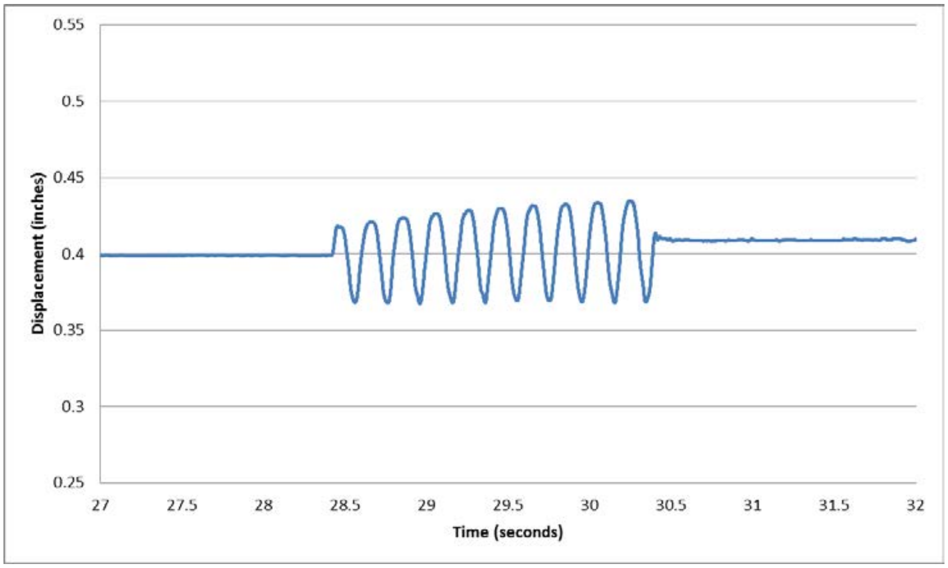


Figure 8. – Cycle showing increase in amplitude.

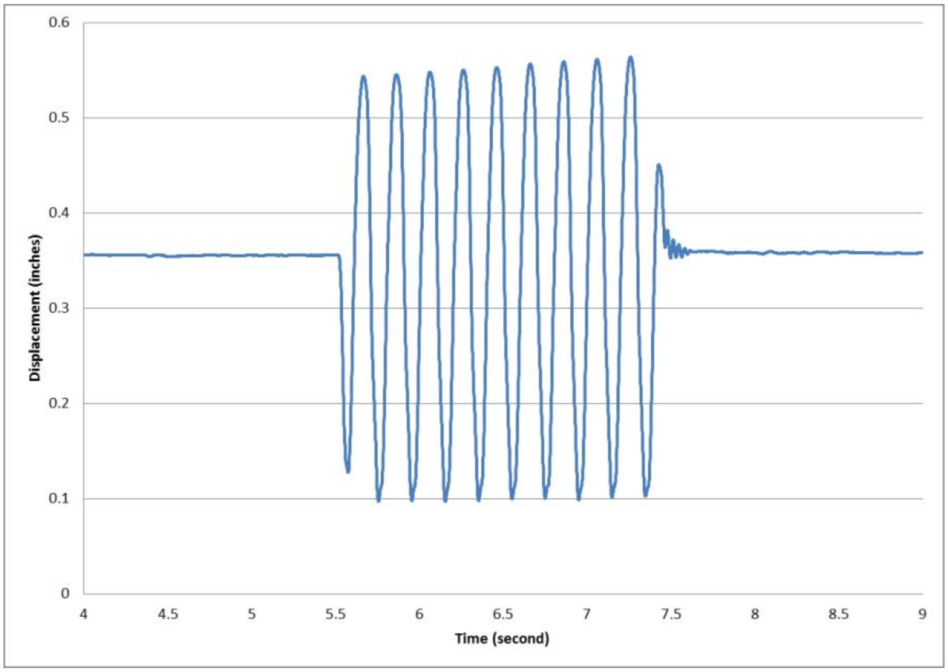


Figure 9. – Cycle showing further increase in amplitude.

As can be seen in **Figure 9**, by this time, the wall is being forced almost back to its original position at the low point of each cycle. In reality, this would be how a spillway wall would respond that is retrofitted with struts. The high point of each cycle would represent the soil pushing the spillway wall into the channel. The low point of the cycle would represent the opposite spillway wall pushing back. Although this pushing back may not force the spillway wall back to its original position, this amplitude could be considered a limiting condition.

In counterforted wall design, the main moment steel is placed in back of the counterfort and is sloped to match the counterfort back face. This report will study in detail how these sloped counterfort rebar take load as the ram displaces the wall. Here, these sloped counterfort rebar are made up of two bars, each bar having three strain gauges along its height near the bottom, where cracking was expected to occur in the experiment. The first set of strain gauges were located about three inches from the bottom of the counterfort (from the top face of the heel). The second and third set of gauges were located 14 inches and 24 inches from the bottom of the counterfort, respectively. Figure 10 shows the locations of these strain gauges. Figure 11, **Figure 12** and Figure 13 show the readings of the three sets of strain gauges. Finally, Figure 14 shows the ram load plotted against displacement of the ram. These figures do not show the cycles involved in each test, but just the beginning value of each test. In the beginning, the cycles did not affect the results, since the amplitudes were small. Toward the end of the experiment, the amplitudes were increased to the point that the hold down mechanism started to displace, which the test was not designed to handle, making the results of this portion of the test unreliable and difficult to replicate numerically. Therefore, this TM will focus on results up to about 0.3 inch of ram displacement only.

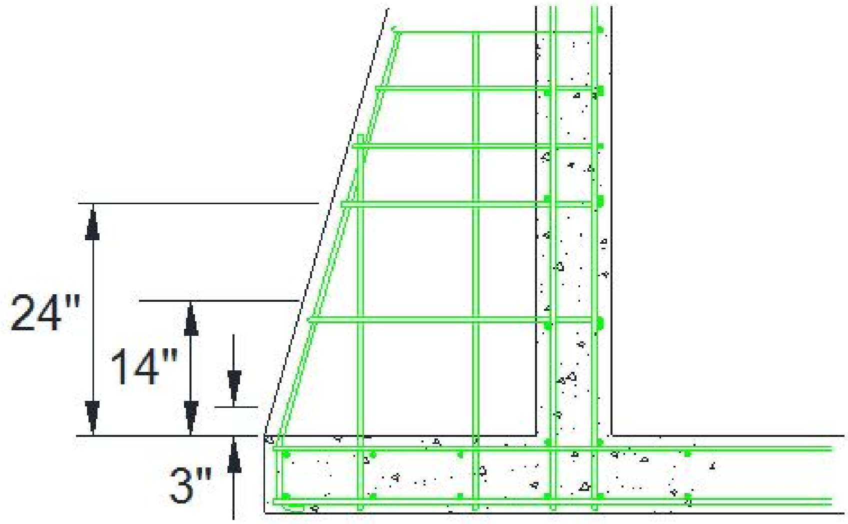


Figure 10. – Location of strain gauges on the sloped counterfort bars.

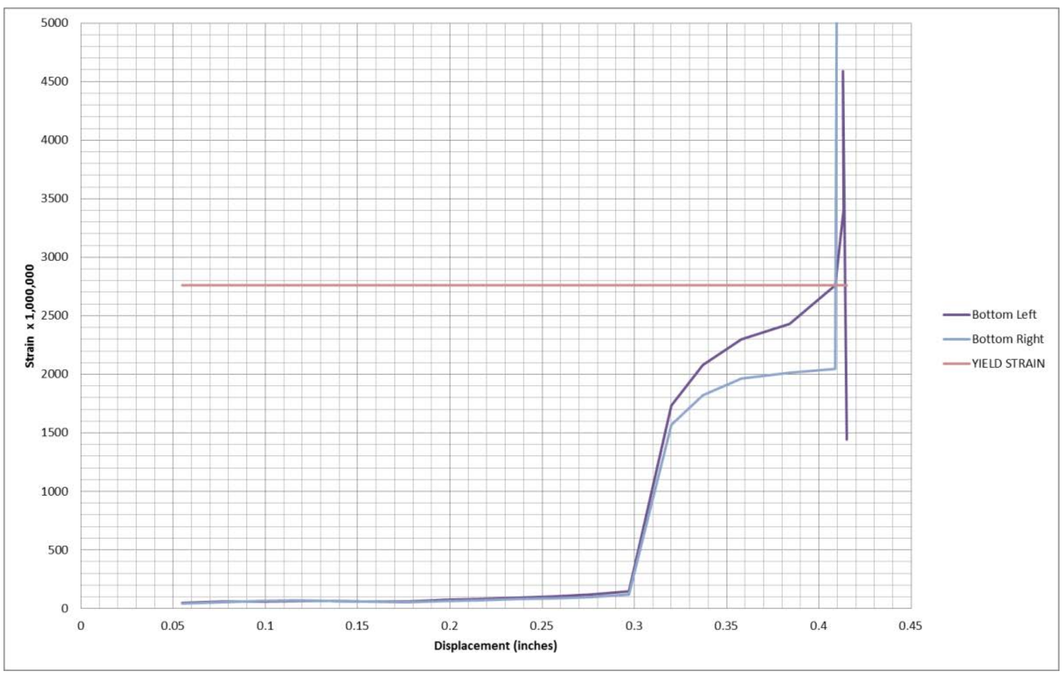


Figure 11. – Strain gauges at 3 inches from bottom.

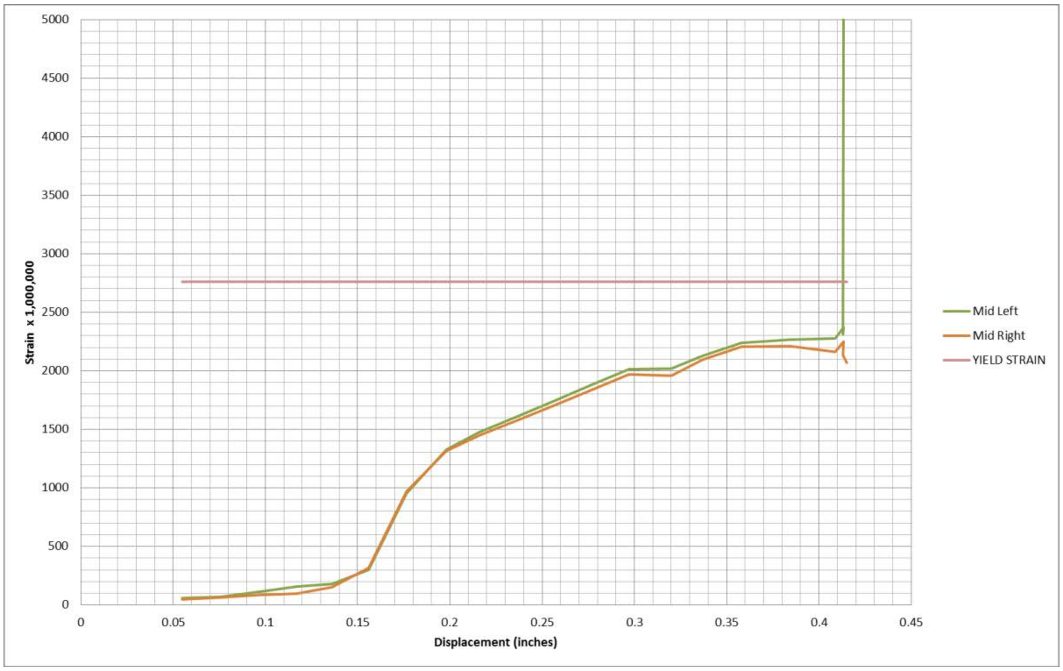


Figure 12. – Strain gauges at 14 inches from bottom.

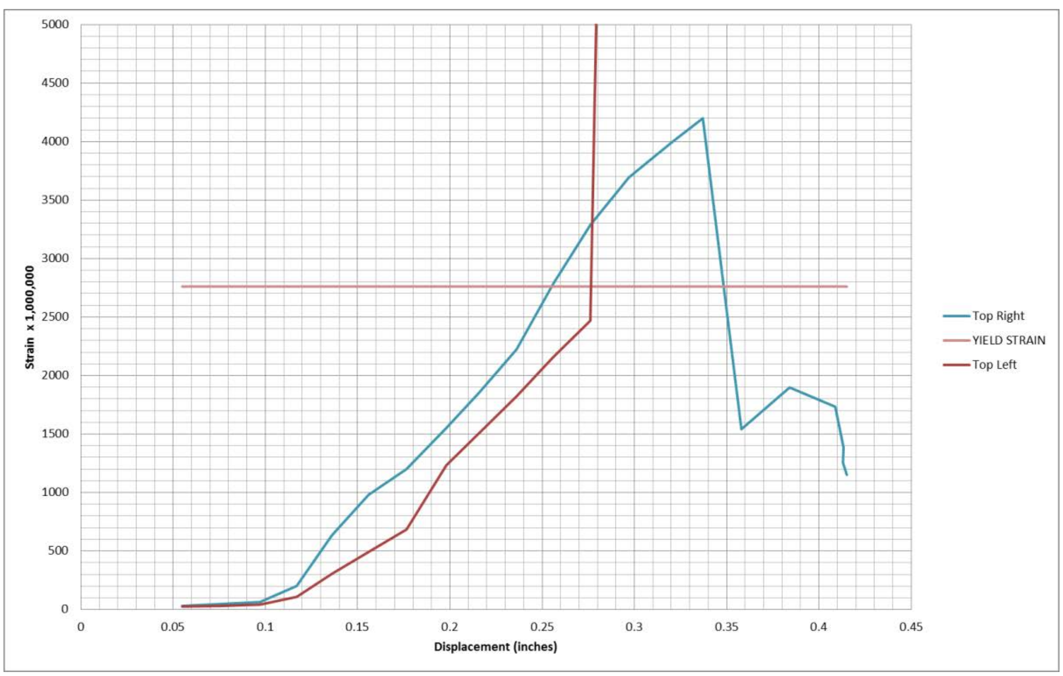


Figure 13. – Strain gauges at 24 inches from bottom.

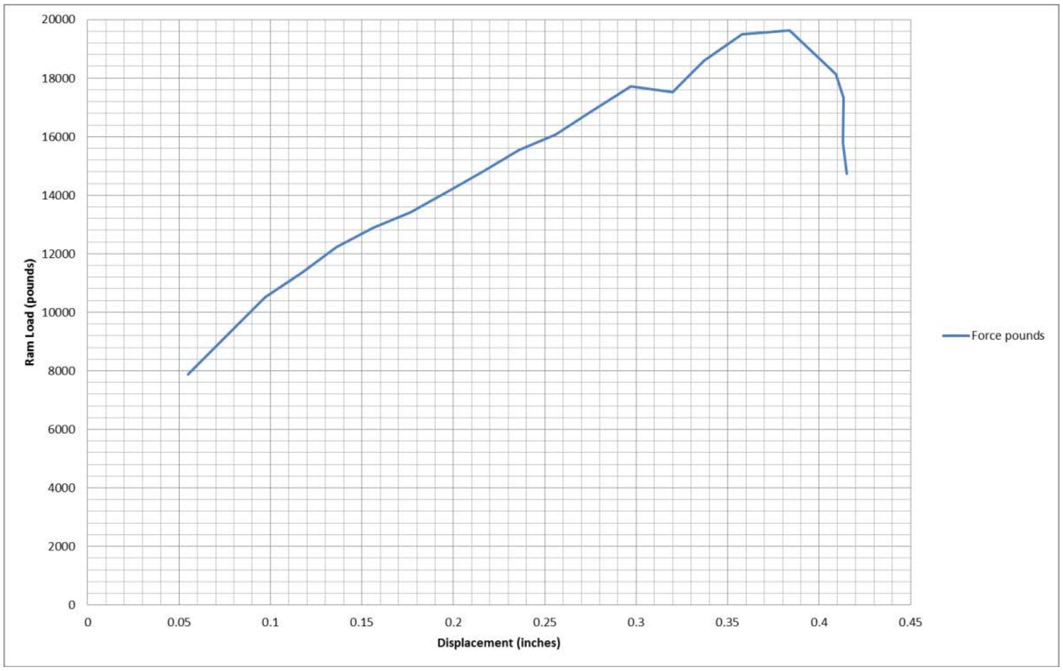


Figure 14. – Ram load versus ram displacement.

Observations of this data include the following:

- Initial cracking occurs first near the top strain gauges as compared with the other locations. This can be seen in Figure 13 where there is an abrupt change in slope at displacement of 0.11 inches. This change in slope occurs when a crack occurs and the concrete transfers stress to the reinforcement in the immediate area. The middle strain gauges exhibit this change in slope later at a displacement of 0.16 inches, while the bottom gauges do not show cracking until a displacement of 0.3 inches.

- The cracking strain of concrete is calculated as follows where the cracking stress is taken as 5% to 10% of f'_c = 2,810 psi:

  $$\text{Cracking strain} = f_{cr} / E_c$$
  $$= 47 \times 10^{-6} \text{ inch/inch to } 93 \times 10^{-6} \text{ inch/inch}$$

  One can see that this is indeed the range where the change in slope occurs with all the strain gauges.

- After cracking, the top gauges maintain a constant slope until past the yield strain of the reinforcement in that area. This is shown in Figure 13. The top left gauge breaks at displacement of about 0.28 inches. This is not to say that the reinforcement ruptured, but just shows a failure of the gauge itself or its attachment to the reinforcement. The constant slope is expected because of the linear displacement loading.

- After cracking, the middle gauges also start with a similar slope as the top gauges. Refer to Figure 12 between a displacement of 0.16 inches and 0.19 inches. However, after this point the slope starts to flatten out. This behavior was not expected. However, this can be explained if one looks more closely where the strain gauge location. Referring to Figure 10, one can observe that at this elevation there is a vertical counterfort bar nearby that can share the load. This bar was not included in the design of the moment steel, but will carry whatever load it is subjected to. The stress in the sloped counterfort steel is reduced because of this, hence a smaller strain than would otherwise occur. This is not the case with the top strain gauges because the influence of the vertical counterfort bar is minimal since this bar does not have the proper development length near this elevation.

- Figure 14 shows how the ram force increases with ram displacement. Again, a near linear relationship exists until a displacement of about 0.3 inches. At this point, another crack forms near the bottom of the counterfort. This can be seen in the change is slope of the strain gauges in Figure 11. After this occurs, the slope of these gauges is similar to that of the middle gauges confirming that once again the vertical reinforcement is sharing some of the load near the bottom also.

- As the loading is increased further, the amplitude of the cycles is now starting to influence the results. Cracking is becoming extreme, with localized rebar buckling near the bottom of the concrete. Any data at this point in the experiment is questionable. The bond between steel and concrete also comes into question later in the test. Spalling around the reinforcement was also witnessed at this time.

III. Finite Element Model

A. Model Details

Figure 15 and Figure 16 show isometric views of the finite element model.

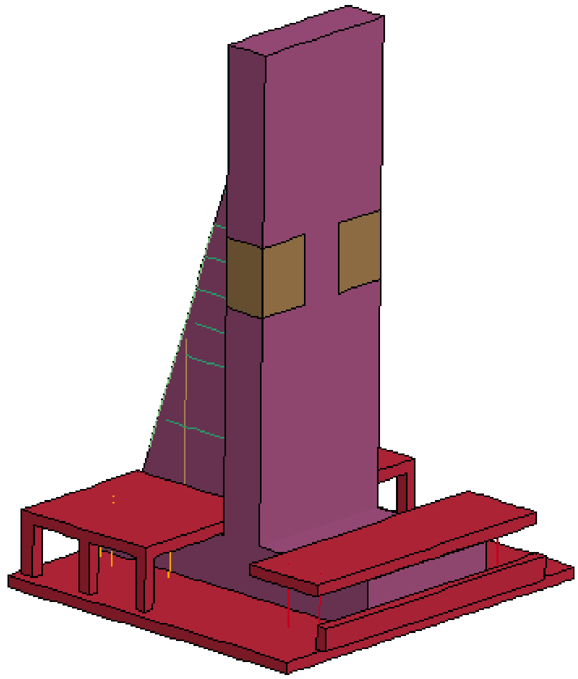


Figure 15. – Isometric view of model.

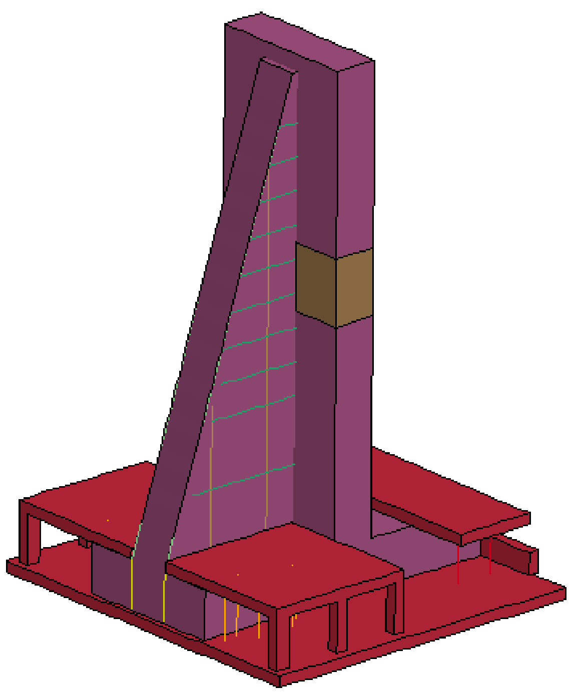


Figure 16. – Isometric view of model.

Figure 17 and Figure 18 show the mesh size of the elements and the reinforcement details for the model.

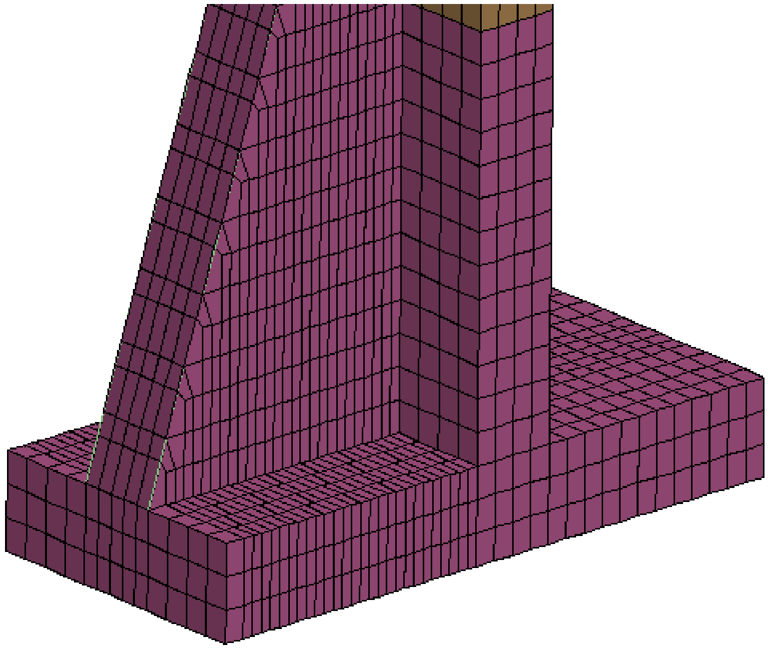


Figure 17. – Mesh details.

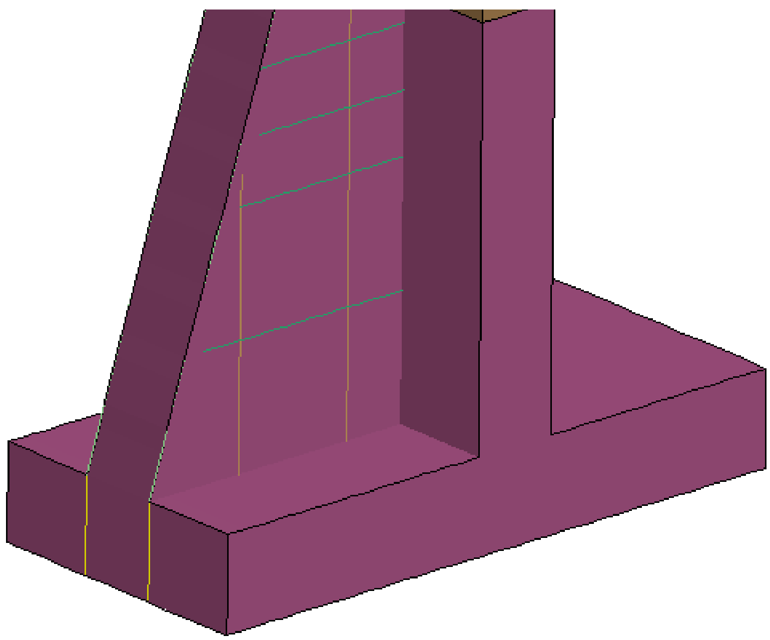


Figure 18. – Reinforcement details.

Truss elements are used to represent the reinforcement. The counterforted wall finite element mesh grid is aligned in such a way that the horizontal and vertical reinforcement along with the sloped reinforcement nodes are coincident with the concrete nodes. Contact surfaces are used between the counterforted wall and the hold down mechanism. These surfaces allow for slip with minimal friction employed. A front bearing plate restricts movement of the wall and the contact force on this plate is a good representation of the ram force applied. The hold down mechanism is anchored down at the heel with truss elements representing the ½ inch diameter bars and at the toe with truss elements representing the 1 inch diameter bars.

B. Concrete

The two LS-DYNA concrete material models will be used in this analysis of the counterforted wall, namely the Continuous Surface Cap Model (CSCM) (MAT 159) and the Karagozian Case Concrete (KCC) (Winfrith). Both the CSCM and the KCC model employ three shear strength surfaces: the yield surface, the limit surface, and the failure surface. The two models differ in their softening evolution equations and in the equations describing degradation of the elastic stiffness (the strain-to-failure is tied to fracture energy release). The KCC model uses an accumulating damage model that adjusts the current strength within any given time step to a stress state varying between the three strength curves used in the model, whereas the CSCM uses Duvaut-Lion visco-plasticity theory to give a smoother prediction of transient effects. Both models support rate dependence by allowing the strength curves to be a function of strain rate.

The CSCM is an elasto-plastic damage material model with rate effects and is equipped with two surfaces: the failure surface and hardening cap. A continuous intersection is maintained between the surfaces. The main features of the model are:

- Three stress invariant yield surfaces
- A hardening cap that expands and contracts
- Plasticity-damage-based softening with erosion and modulus reduction
- Rate effect for increasing strength in high-strain rate applications
- Automated parameter generation scheme based on the uniaxial compressive strength, f'_c

Major advantages of this model are the ability to control the amount of dilatancy produced under shear loading and the ability to model plastic compaction.

The input cards for the CSCM model are as follow (using CONCRETE in the control card, as shown below, requires the user to input only the critical concrete material parameters with all other parameters automatically generated for normal strength concrete):

```
*MAT_CSCM_CONCRETE
1,0.000223,1,0,0,0.95,0,0
0
2810,0.75,3
```

Where:

- Concrete mass density = 0.000223 slugs
- Damage plotting parameter = 1 (show max of brittle and ductile damage)
- Maximum strain increment = 0 (default)
- Rate effects = 0 (no rate effects considered)
- Element erosion = 0.95 (no erosion considered if set to < 1.0)
- Modulus recovery = 0 (modulus is recovered in compression)
- Cap retraction = 0 (cap does not retract)
- Pre-existing damage = 0 (no pre-existing damage)

The KCC material model was developed by Broadhouse and Neilson over many years and has been validated against experiments. Rebar may be included using the smeared rebar approach or by the use of discrete rebar beam elements. For more details on the theory behind this material model, the reader is referred to the LS-DYNA literature. The key features of the KCC model include:

- Three-tiered plasticity surfaces
- Hardening that is related by an Equation of State (EOS)
- Damage based on a damage evolution input curve
- Rate effects for increasing strength in high strain rate applications
- Confinement effects
- Automated parameter generation scheme based on the uniaxial compressive strength parameter, *f'c* (implemented in the third release of the model)

The input cards for the KCC model are as follow:

```
*MAT_WINFRITH_CONCRETE
1,0.000223,3021000.0,0.16,2810.000,140.00,2.0000e-04,0.375
```

Where:

- Concrete mass density = 0.000223 slugs
- Initial tangent modulus =3,021,000 psi
- Poisson's ratio = 0.16
- Concrete compression strength =2,810 psi
- Concrete cracking capacity = 140 psi (or 280 psi used in other parametric runs)
- Crack width = 2.0000e-04 (at which crack-normal tensile stress goes to zero)
- Aggregate radius = 0.375 inches

C. Reinforcement

For following nonlinear reinforcement material will be used:

```
*MAT_PLASTIC_KINEMATIC
2,0.000734,29000000.0,0.3,80000.0,29000.0,0.0
0.0,0.0,0.0,0.0
```

Where:

- Steel mass density = 0.000734 slugs
- Young's modulus = 29,000,000 psi
- Poisson's ratio = 0.3
- Yield stress = 80,000 psi
- Tangent modulus = 29,000 psi

This material describes the reinforcement with an initial stress strain relationship given by the Young's modulus and a final stress strain relationship given by the tangent modulus.

IV. Comparison of Results

Figure 19, Figure 20, and Figure 21 once again show the readings of the strain gauges three inches from the bottom of the counterfort wall, 14 inches from the bottom of the counterfort wall, and 24 inches from the bottom of the counterfort wall respectively. This is the same information as shown in Figure 11, **Figure 12** and Figure 13, but now has the results of the finite element analyses overlayed. Figure 22 has the ram versus displacement information for both the lab results and the finite element results.

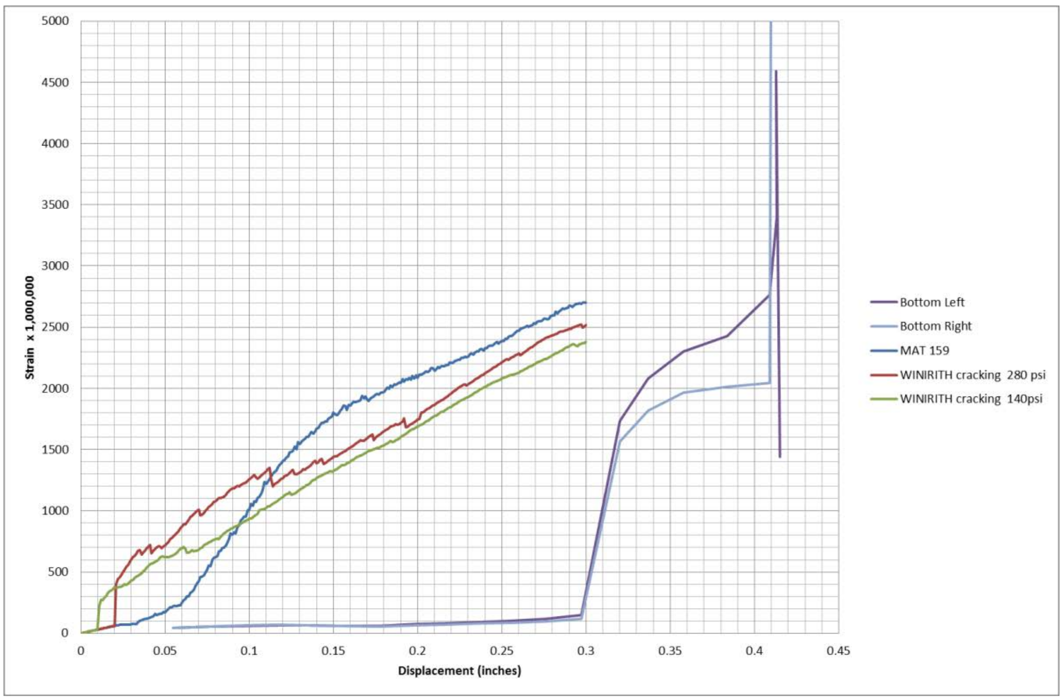


Figure 19. – Strain gauges at 3 inches from bottom.

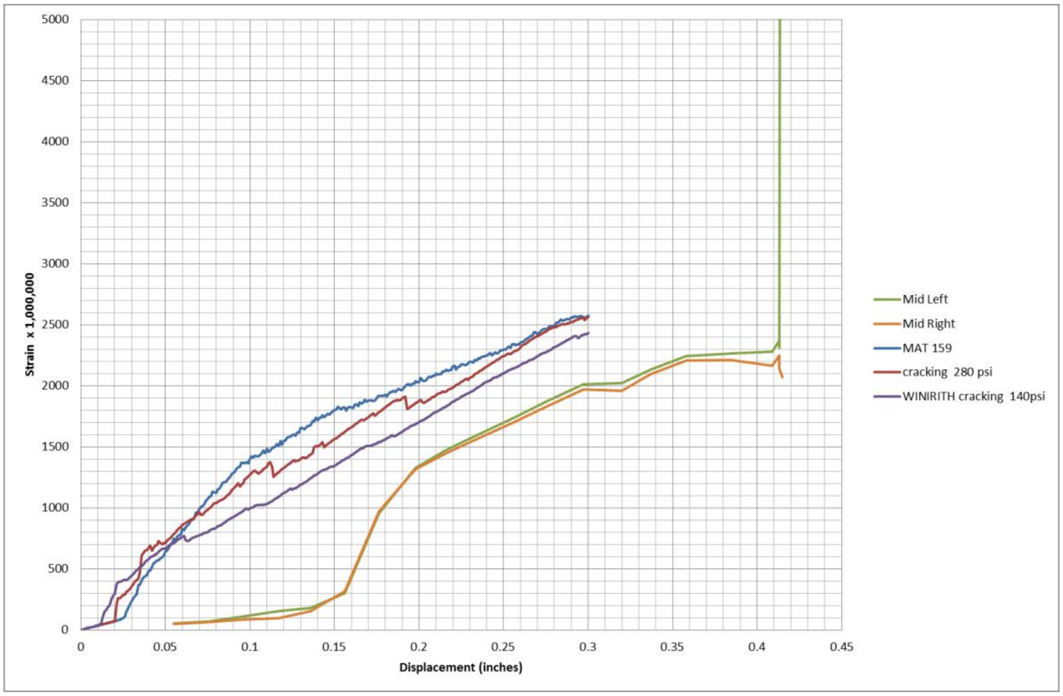


Figure 20. – Strain gauges at 14 inches from bottom.

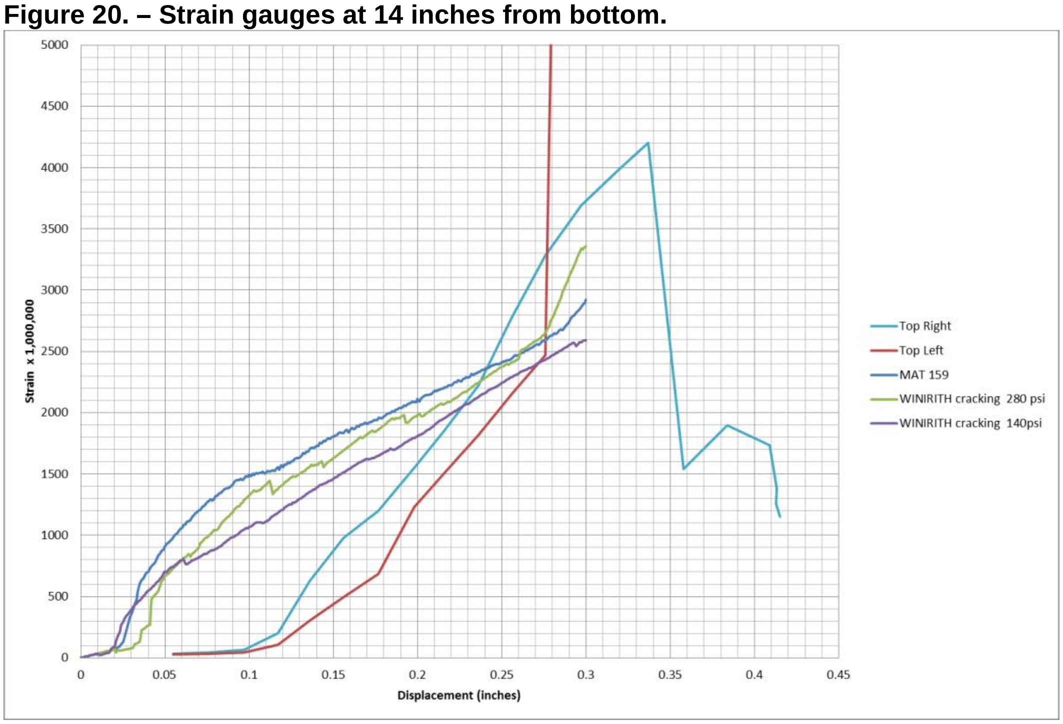


Figure 21. – Strain gauges at 24 inches from bottom.

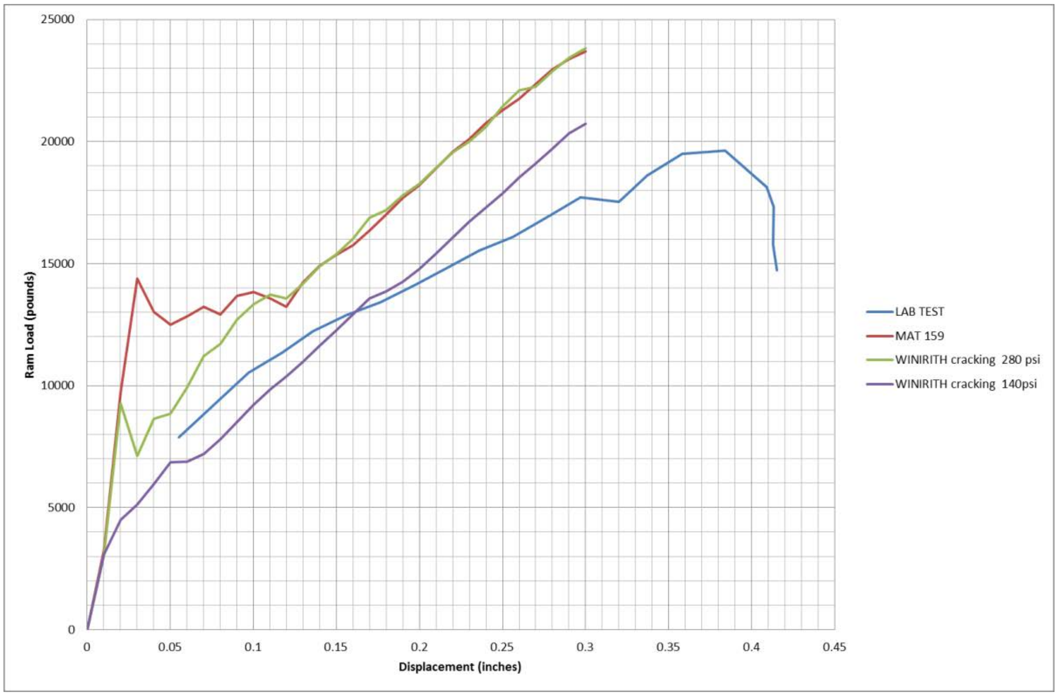


Figure 22. – Ram load versus ram displacement.

Comparison between lab and finite element results include the following:

- Initial cracking occurs much sooner in the finite element analyses as compared with lab data at all locations. The finite element results indicate that the onset of cracking starts around 0.01 and 0.04 inch of ram displacement depending on location and material type chosen to represent the concrete in the program. This is especially true near the bottom of the counterfort wall, where the lab results indicate that initial cracking does not occur until around 0.3 inch ram displacement, see Figure 19.

- As calculated previously, the cracking strain of concrete is:

  $$\text{Cracking strain} = f_{cr} / E_c = 47 \times 10^{-6} \text{ inch/inch to } 93 \times 10^{-6} \text{ inch/inch}$$

  One can see that this is indeed the range where the change in slope occurs with all the strain gauges and with the results of the finite element analyses.

- As stated before, after cracking, the top gauges maintain a constant slope until past the yield strain. This is shown in Figure 21. The top left gauge breaks at displacement of about 0.28 inches. This is not to say that the reinforcement ruptured, but just shows a failure of the gauge itself or its attachment to the reinforcement. The constant slope is expected because of the linear displacement loading. The finite element results also show this same slope at the beginning of the experiment. However, soon the slope changes to a flatter slope. As discussed previously, a change in slope like this indicates that neighboring steel is starting to share in the carrying of the load. The stress in the sloped counterfort steel is reduced because of this, hence a smaller strain than would otherwise occur. This does not occur in the lab results because at this elevation all neighboring bars do not have capacity developed yet. With the finite element model, the bars share nodes and development length is not considered, hence sharing in load carrying can occur much sooner.

- At the middle location, the finite element results show a nearly identical flatter slope as indicated at the top location. The lab results eventually show about the same slope, however, only after a vertical counterfort bar nearby starts sharing in carrying the load.

- Near the bottom, even though the lab results show that the counterfort steel is not stressed until much later, once cracking occurs, the counterfort steel strain quickly increases and the slope flattens out to match the finite element results.

- The above observations suggest that the finite element analyses show a much greater area of cracking occurring sooner with all reinforcement (sloped counterfort, vertical and horizontal shear bars) sharing in carrying the load. The lab results indicate more discrete cracks with less load sharing going on between the sloped counterfort bars and all other bars. One can observe this by comparing the actual damage to the wall in the initial stages of the experiment shown in Figure 23 with finite element results shown in Figure 24, Figure 25 and Figure 26. (NOTE: Figure 23 also shows heavy damage near the bottom of the counterforted wall. This damage occurred later in the experiment when large amplitudes of cyclic loading were applied and the hold down mechanism started to slip). These finite element results also collaborated the general direction of crack orientation.

- Figure 22 shows how the ram force increases with ram displacement. The finite element results indicate a near linear relationship exists until between force and displacement, until the onset of cracking. This onset of cracking depends on the cracking strength input into the finite element model. As can be seen, concrete material Model 159 has a greater capacity then do either of the Winfrith material models with cracking set to 140 psi or 280 psi (5% or 10% of f'_c, respectively).

- Toward the end of the experiment, the finite element model indicates that it takes more ram force to displace the wall a certain amount as compared with lab results. However, as mentioned above, the finite element results also indicate more damage to the counterforted wall. This is contrary to the belief that it takes less force to deflect a more damaged structure since the stiffness is reduced as damage increases. Part of this discrepancy may be due to the fact that the finite element model does not model localized concrete/reinforcement interaction. A finer mesh may resolve some of this issue.

- One parameter with respect to single reduced-integration finite elements is the proper use of hourglass control. This is an artificial restraint imposed on an element to help deal with non-stiffness deformation modes. Various hourglass controls were implemented with the best results obtained using a stiffness based approach (hourglass = 5) with a coefficient set to about 0.1.

V. Conclusions

A scaled model of a counterforted wall was tested in the laboratory using a loading ram with displacement control to simulate earth loads. Strain of the counterfort moment steel was obtained along with ram load and displacement of the wall. Cracking patterns were documented. Finite element analyses of this wall were also done as part of an ongoing effort to validate the use of finite element codes to predict the behavior of these walls.

Figure 23. – Actual damage to counterforted wall.

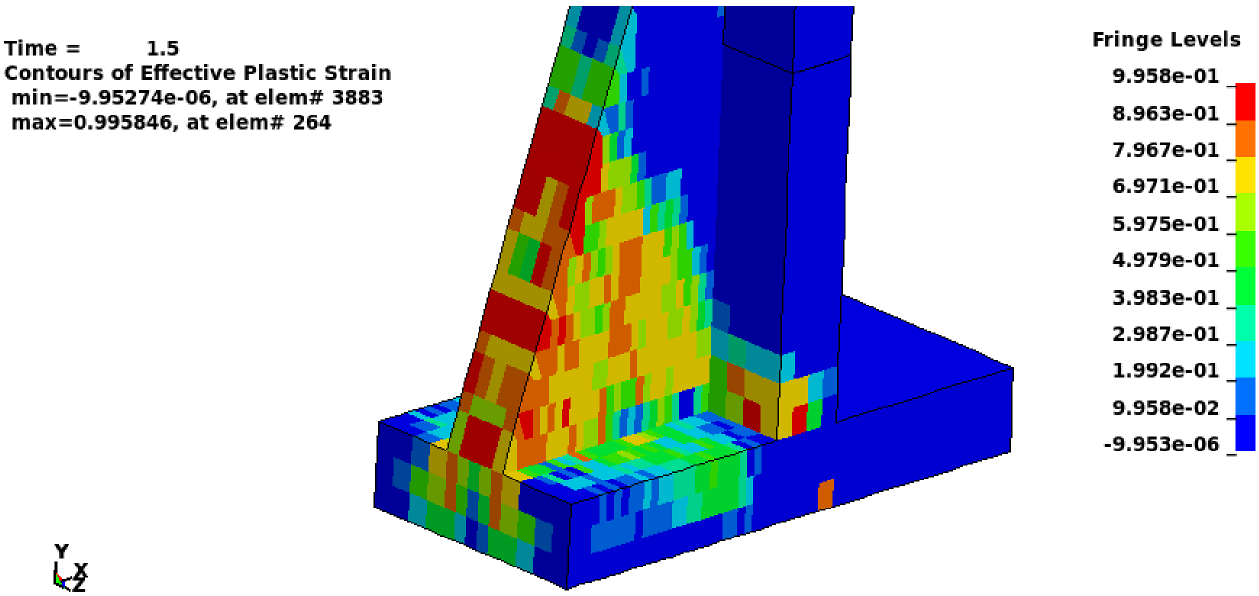


Figure 24. – Damage to counterfort wall (MAT 159).

Figure 25. – Damage to counterfort wall (Winfrith with cracking 280 psi).

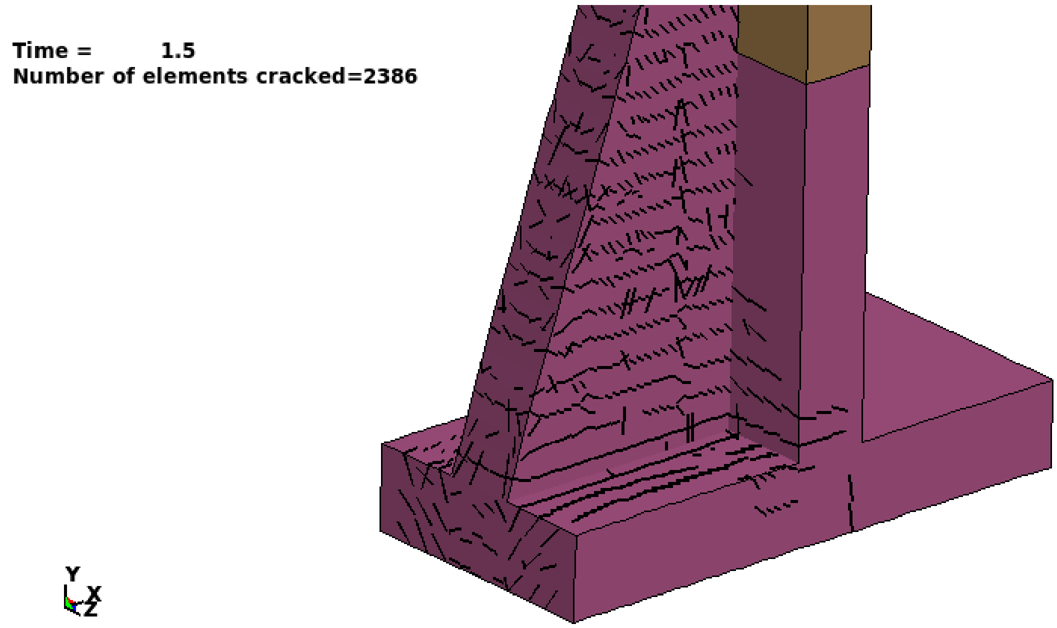


Figure 26. – Damage to counterfort wall (Winfrith with cracking 140 psi).

In general, the finite element analyses show a much greater area of cracking occurring sooner with all reinforcement (counterfort moment steel, vertical and horizontal shear bars) sharing in carrying the load. The lab results indicate more discrete cracks with less load sharing going on between the counterfort moment steel and all other bars.

The finite element analyses supported the locations of early cracking along with crack orientation.

The finite element model indicates that it takes more ram force to displace the wall a certain amount as compared with lab results. However, the finite element results also indicate more damage to the concrete in the counterforted wall. This is contrary to the belief that it takes less force to deflect a more damaged structure since the stiffness is reduced as damage increases. Part of this discrepancy may be due to the fact that the finite element model does not model localized concrete/reinforcement interaction. A finer mesh may resolve some of this issue.

VI. Recommendations

Finite element model results indicating that it takes more ram force to displace the wall a certain amount as compared with lab results is not conservative. This would mean that the finite element results show that the capacity of spillway walls is greater than actual. Therefore, it is recommended that a second scaled model of the counterfort wall be constructed (same dimensions and rebar pattern) and loaded similarly in order to have a better statistical sample. This would eliminate the possibility of error in lab results and give more data to verify finite element modeling.

As far as the modeling of the wall, it is recommended that a closer look be taken as to how the reinforcement is coupled with the concrete. Adjustments to account for development lengths and localized concrete/reinforcement interaction could be made.

Constructing a second scaled counterforted wall would be efficient since the labs currently still have the formwork and the expertize in building this wall.

References

[1] "LS-DYNA – Version 971," Livermore Software Technology Corporation, 2876 Waverly Way, Livermore, California, 94551.