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SEISMIC DESIGN OF A MAJOR PRECAST CONCRETE STRUCTURE IN THE NEW MADRID SEISMIC ZONE

J. E. Beavers[1] and T. D. Al-Shawaf[2]

ABSTRACT

This paper provides an overview for the seismic design of two Department of Energy (DOE) depleted uranium process facilities, one in Paducah, Kentucky, and one in Portsmouth, Ohio. The purpose of these two projects is to convert the uranium enrichment byproduct left over from the enrichment process of the "Cold War" days to a more stable form for reuse and or disposal. Per the DOE design criteria these facilities had to be designed for earthquake loads. This paper primarily focuses on the facility being built in Paducah, Kentucky, because it is in the northeastern part of New Madrid Seismic Zone, the highest seismic zone in the eastern United States.

Introduction

The authors were peer reviewers for the seismic design of two Department of Energy (DOE) depleted uranium hexafluoride conversion projects (DUF6) at Paducah, Kentucky and Portsmouth, Ohio. The purpose of these two projects is to convert the uranium enrichment byproduct left over from the enrichment process of the "Cold War" days to a more stable form for reuse and/or disposal. The design and construction of these projects was awarded to Uranium Disposition Services (UDS), LLC in 2002. UDS comprises three companies: Energy Solutions, Inc. (formerly Duratek Federal Services, Inc.), Burns and Roe Enterprises, Inc., and AREVA Federal Services (AFS), Inc. Burns and Roe Enterprises, Inc. is providing architect-engineering, procurement, and construction management services. Energy Solutions provides project operations and waste management experience. AREVA NP provides commercial uranium processing technology based on similar AREVA facilities now operating in Richland, Washington and Lingen, Germany.

For DOE the independent design and construction oversight of these projects was provided by Oak Ridge National Laboratory and its subcontractors. They provided technical support to the group known as the integrated project team (IPT). In addition, DOE Standard 1020 (DOE 2002) requires a peer review of the seismic design within the framework of a graded approach with increasing levels of rigor for the structures systems and components (SSCs). This review group (the authors) was known as the Independent Seismic Review (ISR) team and was a subset of the IPT. The ISR team did not get involved in the project until the designs were well underway, i.e., the design process on the projects started shortly after UDS received the award, however, the ISR team did not have its first meeting with UDS until April 20, 2005.

[1] Principal, James E Beavers Consultants, jbeavers@jebconsultants.com, Knoxville, Tennessee, USA.
[2] Engineering Manager, Structural and Engineering Mechanics Group, AREVA, Taha.AlShawaf@areva.com, Naperville, Illinois, USA.

The Conversion Facilities

The main subject of this paper is the large conversion buildings shown in Figure 1, which are the main portion of each of the DUF6 conversion projects. The aspect of the buildings are approximately 1 wide to 1.5 long with a high bay corridor running through the approximate center of the structure as shown in Figure 1. The conversion buildings comprise five roof levels. Level one, the lowest level is at Elev. 1[3] (above grade), level two is Elev. 2, level three is at Elev. 3, level four at Elev. 4, and level 5 (the high bay area) is at Elev. 5. Because of these various levels and corresponding shapes, it was concluded by the authors that the conversion facility would have to be considered as irregular in plan and vertical for seismic design based on Tables 1616.5.1and 1616.5.2 of the International Building Code (IBC) (IBC 2000).

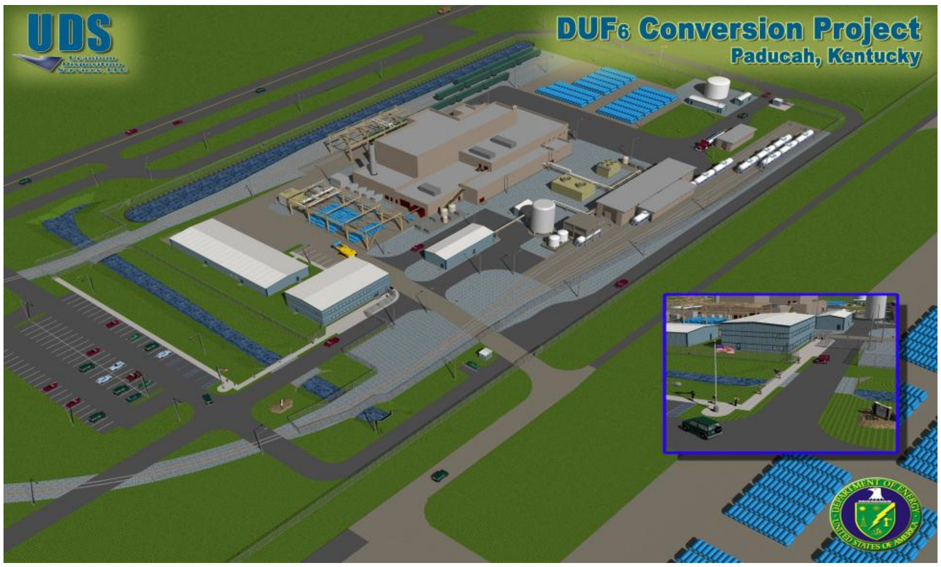

Figure 1. Architectural rendering of the DUF6 Conversion Project

Seismic Performance Category

As required by DOE Order 420.1 (DOE 1995) and the corresponding Guide (DOE 2000) a graded approach is used in which natural phenomena hazard (NPH) requirements are provided for various performance categories, each with a specified performance goal. DOE Standard 1021 (DOE 1993) provides the guidance used for assigning NPH performance categories for structures, systems and components (SSCs). Using this guide, the bulk of the DUF6 facilities were assigned Performance Category (PC) 2 by UDS, including the building structures, the objects of this report. Since the DUF6 SSCs were categorized mostly as PC 2 facilities, that allowed the projects designers to use the IBC for most of the designs, i.e., the design code of record.

Seismic Design Criteria and Approach

Criteria

As stated, the code of record for the DUF6 facilities was IBC 2000. The IBC 2000 used the 1996 USGS seismic hazard maps (Frankel, et al. 1996). In addition, Beavers (2005) and Kennedy (2005) conducted independent assessments of seismic hazard criteria that should be

[3] The exact roof elevations are not given because of the information being Official Use Only (UOU) by DOE.

used for the DUF6 facilities for both the Portsmouth, Ohio and Paducah, Kentucky sites. Later Beavers and Kennedy (2005) recommended the seismic design criteria for the DUF6 SSCs use the USGS 2002 maps (Frankel et al. 2002) rather than the USGS 1996 maps (Frankel et al. 1996).

During the 1970s through the 1990s DOE had a number of site-specific seismic hazard studies conducted to determine the seismic hazards at both Paducah and Portsmouth. The most recent was by Risk Engineering for Paducah in 1999 (REI 1999). As a result, while the Portsmouth seismic hazard was taken directly from IBC, the seismic hazard for Paducah was reduced 20 percent per Section 1515.24 of the IBC. In addition, since Paducah, Kentucky is located near the north-east corner of the New Madrid Seismic Zone (NMSZ), the remainder of this paper will focus on the conversion building at Paducah, Kentucky. The seismic design spectra for the Paducah site is shown below in Figure 2 with the middle curve in the 0 to 0.8 sec time frame as the design spectra and the upper curve in the 0.8- to 2.0-second time frame.

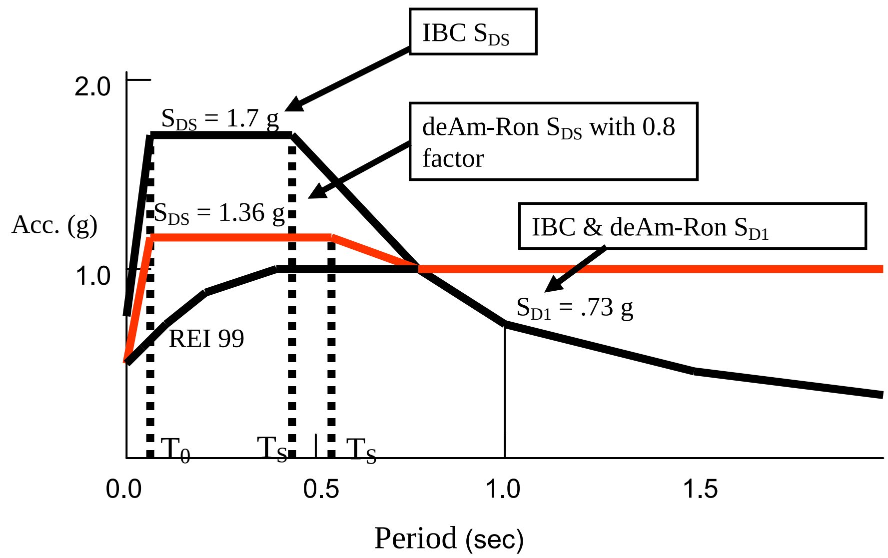

Figure 2. Response Spectra for the Paducah Site

Equivalent Lateral Force (ELF) Procedure

Using the ELF procedure is acceptable for buildings that are regular in plan and vertical. However, due to the conversion building being both irregular in both plan, and vertical (as shown in Figures 3 and 4), the ISR team had some concerns about using the ELF procedure. It was not clear whether the ELF procedure was used for the designs until a second meeting was held with UDS on December 22, 2005. At this point in the project, designs of the DUF6 conversion buildings were basically complete and construction had begun at Portsmouth and was ready to start at Paducah within a few weeks. To avoid shut down of the projects, the ISR team agreed to reevaluate the need for a dynamic modal analysis of the conversion buildings based on the irregularity issue. It was also agreed that if the designers had a well-defined load path from the top down through the structure and demonstrated that the precast concrete would emulate cast-in-place concrete per the American Concrete Institute publication (ACI 2001), which was not in the original design criteria - the seismic design would be approved by the ISR team.

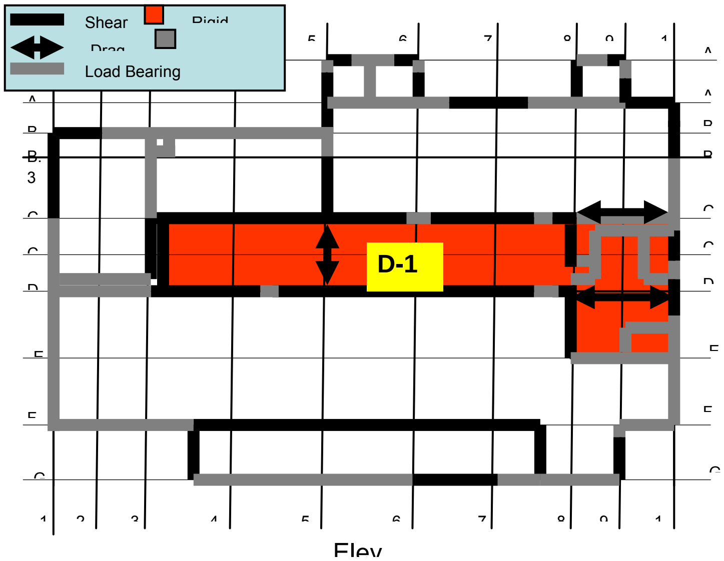

Figure 3. Diaphragm 1

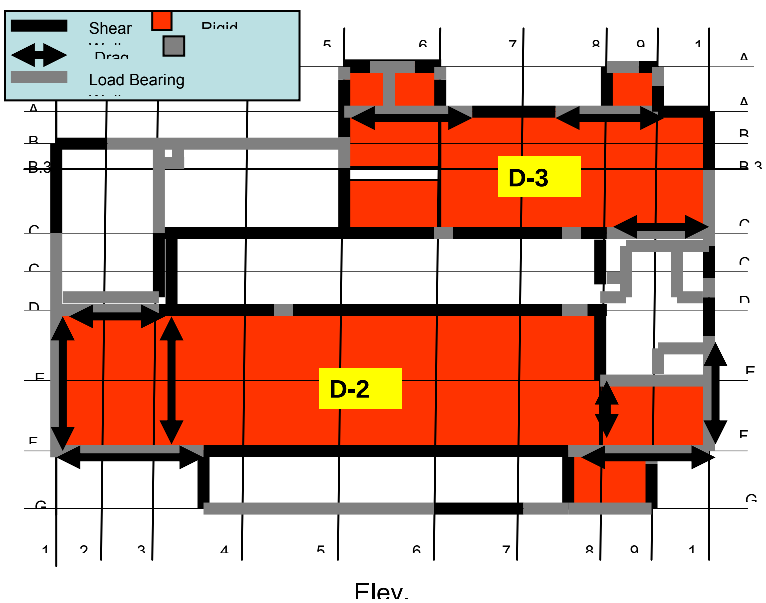

Figure 4. Diaphragms 2 and 3.

To support the use of the ELF procedure for the irregularity of the conversion buildings, the ISR team conducted an independent study as part of its seismic assessments (Beavers and Al-Shawaf 2007 and 2009). They concluded that for the irregularity issues with the conversion buildings the ELF procedure should work well. This was later supported by dynamic analyses (Beavers and Al-Shawaf 2009).

Conversion Building Analysis and Design

To conduct the analyses and designs of the conversion buildings, UDS hired local contractors. At Portsmouth, UDS awarded a contract to Consulting Engineers Group, Inc. (CEG) and at Paducah to deAM Ron Building Systems. At Paducah, deAM Ron Building Systems subcontracted the seismic analysis of the conversion building to Maurer Structural Engineering (MSE), LLC.

In both cases, Paducah and Portsmouth, the building material of choice, as alluded to above, was precast concrete. Figures 3 and 4 shows the reader some of the irregularity issues with major diaphragms being in significantly different locations within the building plan. To provide conservatism in the design the project team decided to ensure that in the design earthquake the building diaphragms at both Paducah and Portsmouth would remain in the elastic range, thus any earthquake energy absorption would occur in the shearwalls. As a result, the capacity-demand (C/D) ratios for the diaphragms were set at 2.0 or greater even though a Response Modification Factor (R) equal to 5.5 was used in the design.

Analysis and Design

The discussion in this section is based on the ELF method of analysis (UBC 2000).

Base Shear

Using the ELF procedure MSE calculated the base shear using IBC Equation 16-34, i.e., $V = C_SW$ where C_S is a seismic coefficient and W is the weight of the building. The ISR team conducted some independent checks of the analysis process, in addition to the seismic hazard assessments discussed above. For example, the ISR team value for the fundamental period of the conversion building was 0.415 sec while MSE's value was 0.42 sec. Later MSE's computer model obtained a fundamental mode of 0.176 seconds while CEG's obtained a T of 0.1811 sec for the conversion building at Portsmouth (Beavers and Al-Shawaf 2007).

The response modification factor is a function of seismic-force-resisting system which, as noted above is precast concrete. As noted in Beavers and Al-Shawaf (2009), the ISR team wanted the precast concrete used as a seismic-resistant system in building construction to emulate special reinforced concrete shear walls with steel elements. In addition, there were two types of shear walls within the building, those that are infilled walls such as the high bay area along grid lines C and D (frames with shear wall between the columns) and load-bearing walls as in grid lines F and G (Figures 3 and 4). As a result, per Table 1617.6 the infilled walls system should have an R = 6.0 (Item 2E or 5C of Table 1617.6 9 (IBC 2000)) and the load-bearing shear walls will have an R of 5.5 (Item 1B of Table 1617.6 (IBC 2000)). In the analysis, MSE used an R of 5.5 (Beavers and Al-Shawaf 2009). Since most of the shear load is carried by the walls in the high bay area grid lines C and D (Figures 3 and 4), it is reasonable to select an R value of 6.0.

Using an R of 6.0 an I_E of 1.5 and the authors S_{DS} of 1.36 g from the USGS 2002 maps, Equation 16-35 results in a C_S of 0.340. From Equation 16-36 and $S_{D1} = 0.73$, using a T of 0.415, results in a C_S of 0.439 and a T of 0.176, in a C_S of 1.037, and from Equation 16-37 C_S is calculated as 0.089. Thus, the C_S value calculated (0.340) is smaller than the two values calculated using Equation 16-36 and larger than that calculated using Equation 16-37. Thus, the $C_S = 0.340$ can be used to calculate the base shear. MSE used a C_S of 0.371 (deAM-Ron 2005). Compared to the authors' value this adds approximately nine percent conservatism (high bay area only) to the base shear. This conservatism is the result of MSE's using a response modification factor I of 5.5 rather than 6. With a C_S of 0.34 and added conversion building weight to the MSE model (deAM-Ron 2005) the authors obtained total building weight of 16665 kips resulting in a base shear of 5666 kips compared to MSE's 6181 kips.

Load Path

The authors then reviewed the lateral load calculations provided by deAM-Ron (2005). The major contribution of seismic load in the building is a result of the rigid diaphragms. The seismic loads induced at the diaphragms were computed as 1767 kips at the roof level, 594 kips at Elev. 6[4], 764 kips at Elev. 5, 635 kips at Elev. 4, 801 kips at Elev. 3 with 576 kips at D-4 and 225 kips at D-5 (Figure 5), 1242 kips at Elev. 2 with 719 kips at diaphragm D-2 and 523 kips at diaphragm D-3 (Figure 4) and 378 kips at elevation 1 (D-1, Figure 3 and 5) for a total base shear of 6181 kips. The seismic loads are also shown in a math model of the conversion building in Figure 5.

The authors were able to trace this base shear from the diaphragms to the foundation with total shear numbers in the east-west direction adding up to be 5824 kips and in the north-south direction 6250 kips. These numbers were not exact, but were close enough to the 6181 kip diaphragm seismic load to verify the load path is traveling appropriately through the shear walls using the ELF procedure. As noted above, this was later verified with the dynamic analysis conducted later in the project.

Capacity vs Demand

Another validation that the ELF procedure is adequate for analysis of the building is the amount of conservatism in the design. As noted in Beaver and Al-Sawaf (2009), the base shear used in the design of the building has a potential additional seismic load of 44 percent, or an actual seismic load of nine percent.. During the design process the C/D ratio was checked by the authors in all of the shear walls (SW). This was a key fundamental aspect of understanding the building's performance during the design seismic event. During the review process the first author found some apparent deficiencies in the C/D ratios of some key walls i.e., the C/D ratios for Shear Walls (SW) 4 and 7 were less than 1.0. The readers are referred to Beavers and Al-Sawaf (2009) for the final assessment of the C/D ratios of the shear walls.

Shear Between Vertical Panels

The authors then verified the connections between vertical precast panels to assure the shear was being taken downward through the system into the foundation. For construction the panels making up the shear walls in grid lines C and D were placed horizontally, while nearly all other shear-wall panels were placed in a vertical position (deAM-Ron 2005a). As an example, along grid line G, between gridlines 6 and 7, SW 16 was the only shear wall in gridline G. From the computer analysis the shear in SW 16 is 237.93 kips (deAM-Ron 2006). Panels W12-69 through W12-71(deAM-Ron 2005a) basically made up SW 16, plus about half of panel W12-72, i.e., approximately 3.5 panels. Thus, on the conservative side, the individual panel shear will be about 68 kips. As a result, this load must be transferred down through the panel to panel connectors at the foundation.

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[4] The exact mass elevations are not given because of the information being Official Use Only (UOU) by DOE and do not necessarily correlate with roof levels.

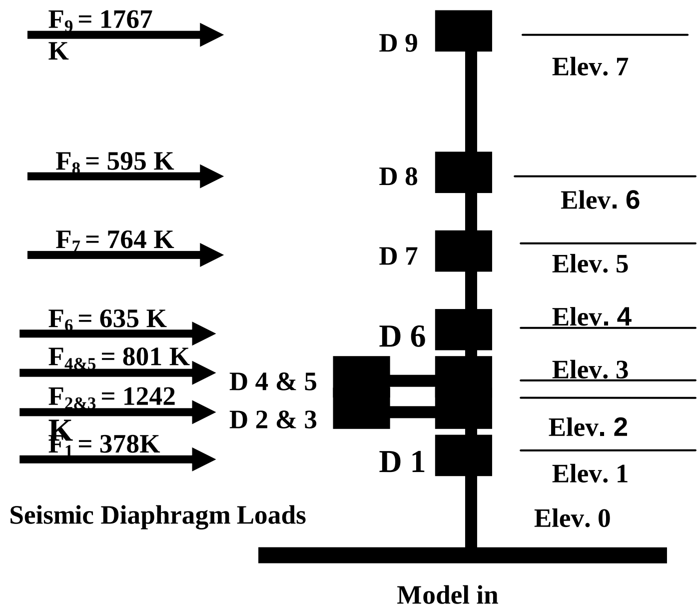

Figure 5. Vertical Mass Model of the Conversion Building

Shear at Foundation

During the review the authors found most of the precast concrete shear walls had plenty of capacity verses demand with only four shear walls out of 37 having a C/D ratio of less than 2.0. However, the weak link in precast concrete construction is a function of the connections. The interface between the wall and the foundation has two connections, namely the wall-embed plates and the baseplate in the foundation. These two connections were welded to each other. The capacity of this joint is the minimum capacity of the wall-embed plate, the base plate and the weld joining them. The numbers of shear wall connections (wall-embed plates) to foundation base plates vary with shear wall, both in number and strength. Most of the wall-embed plates are "WP-59" type while most of the base plates are identified as "CP-7," and are used in the bulk of the shear-wall-to-foundation connections. For example, shear wall SW-1 has six "WP-59" and six "CP-7" shear plates, each with capacity of 74.66 kips and 71.06 kips, respectively. Note that there are two capacity values for CP-7; i.e., 71.06 kip if located near "Free Edges" and 103.38 kips if located elsewhere (Uranium Disposition Services 2007, Appendix B, pg. 12 and 14). Hence, for SW-1 the total shear-wall-embed-plate capacity of 447.96 kips, and a total of 426.60 kips for baseplate capacity, while the plate-to-plate weld capacity is 1238.24 kips. The minimum of these values is 426.60 kips which is the connection capacity for SW-1. This process is repeated for all shear walls noting the number and different designs of the embed plates and base plates. However, the weak link in these connection locations is the connections in SW-37 wall which has only a total of 351.54 kips of capacity with a C/D ~1.0. The shear wall SW-7 had one of the lower C/D (1.08) ratios in the original design (deAM-Ron 2005b) not considering connections. The connections for SW-7 have a total capacity of 942.48, which results in a C/D ratio of 1.21. Thus, the shear wall itself is the weak link here with only a capacity of only 837 kips, and will yield as expected during the design earthquake levels.

Comparing all of the shear wall capacities with the shear-wall plate capacities, SW-7 is the one of only two walls where the shear-wall plate capacity (connections) is higher than the shear wall. As a result, for most cases the shear-wall plate C/D ratios are smaller than the shear wall C/D ratios. For the shear walls, only four had a C/D ratio less than 2.0 while there are 23 shear wall connections having a C/D ratio less than 2.0. The lowest C/D ratio for the shear wall plates is at SW 37 with a C/D ratio of 1.0. Thus, if the design earthquake occurs many of the shear plates will go nonlinear. However, as discussed above the potential conservatism will increase the C/D ratio. The discussion in this section was based on the static analysis of the building and is revised further using the dynamic analysis results discussed below.

Overturning and Out-of-Plane Loads

Gross overturning moments of the building are represented by the distribution of the base shear along the height of the building that is equal to 212,102 kip-ft. An overturning capacity of the building was calculated by UDS (UDS 2006). This analysis showed an overturning moment capacity of 1,547,324 kip-ft. and along with the above gross overturning moment of 212, 102 kip-ft. results in a C/D ration of 7.3.

The authors conducted some checks on the out-of-plane loads for verification. In the case of wall panel W 16-39 along grid line C, between grid lines 6 and 7, the out-of-plane force was 125 lbs per sq. ft. resulting in a maximum moment of 126.3 kip-ft. (deAM-Ron 2007) with the ultimate moment capacity of the panel 131.9 kip-ft. Thus, as one check, this verifies that the panels should have the capacity to carry the out-of-plane loads.

Dynamic Analysis

As part of the independent review process in April 2007 representatives of the IPT and ISR teams went to the AREVA NP office complex in Charlotte, N.C. to conduct a status review of the DUF6 design progress (Beavers and Shawaf, 2009). At this time the first ETABS analyses has been conducted and the result compared against the ELF procedure. During this review the ISR team calculated the C/D ratios for SWs 4, 7 and 8. They found C/D ratios for SWs 4, 7 and 8 as 0.86, 0.82 and 1.15, respectively. As a result of these findings and others that were identified independently by UDS, UDS created an engineering design team (UDS-EDT) to assist deAM-Ron in the final phases of the design process.

The purpose of the UDS-EDT was to re-evaluate the demand on the shear walls, make appropriate modifications to the shear walls as required based on the demand loads resulting from changes that occurred to the diaphragms, e.g., additional topping added to the diaphragms. UDS-EDT made a decision to use the software package ETABS (2006), instead of Risa3D as used in the original model analysis by deAM-Ron to conduct a re-analysis of the process building.

In the case of the shear walls as noted above, SW 4 had a C/D ratio of 0.86; as a consequence, the thickness was increased from foundation elevation 99' to elevation 127' by 12" resulting in a 28-inch thick wall from the original 16-inch wall thickness used in the ELF procedure. The ETABS analysis demand on the shear wall at elevation 99 was 1431 kips. The additional thickness of SW 4 resulted in a capacity of 3041 kips UDS (UDS 2007). Thus the C/D ration for SW 4 at the foundation is 2.12 from the ETABS analysis, well above the required 1.0. At elevation 127', where the additional thickness to SW 4 ends, the capacity of the original SW 4 was1810 kips. The maximum shear load based on the RISA analysis just above elevation

127 was 789 kips (Beavers 2007); therefore, result from elevation 127 to 159, the C/D ratio of the upper portion of SW 4 is 2.3.

In the case of SW 7, instead of increasing the thickness, SW 7 is in line with SW 9 and SW 9 will carry the increased load in SW 7 resulting in a C/D ration in SW 7 of 1.0 or above. SW 8 (mentioned above) already had a C/D ratio of 1.15 based on the simplified review in April of 2007 (Beavers and Al-Shawaf, 2009). Based on the ETABS analysis the demand on SW 8 was 728 kips, smaller than the RISA analysis results of 859 kips. So the SW 8 C/D ratio was well over 1.0.

Conclusions

The authors reviewed the seismic design of the DUF6 conversion facility at Paducah, Kentucky and found that, except for a few minor issues, discussed above the facility has more than enough capacity to resist the design earthquake. The Equivalent Lateral Force (ELF) procedure was found adequate for the task. However, during the project a dynamic analysis using ETABS was conducted on the facility. This analysis was conducted as a result of design changes that occurred during construction as discussed in the text. The ETABS analysis supported the ELF procedure. In addition, the diaphragms were designed to remain elastic, with a capacity/demand (C/D) ratio higher than 2.0, even though a Response Modification Factor (R) equal to 5.5 was used in the design. The lowest C/D ratio in the shear-wall, lateral-resistant system was 1.0 and in some cases as high, or higher, than 2.0. The lowest C/D ratio for overturning moments was found to be 7.3. It is, therefore, likely that if the design earthquake occurs many of the structural components of the conversion building will remain in the elastic range, well above that required in a normal building.

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