San Francisco-Oakland Bay Bridge Seismic Retrofit Project

Independent Review of Analysis and Strategy to Shim Bearings at Pier E2 to Achieve Seismic Design Requirements

Modjeski and Masters, Inc.

August 9, 2013

BAY AREA TOLL AUTHORITY

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Table of Contents

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Executive Summary and Conclusions

Modjeski and Masters, Inc. (MM) was retained to conduct a peer review of the suitability of the "Shim Concept" to provide sufficient seismic capacity at Pier E2 of the San Francisco-Oakland Bay Bridge (SFOBB) to withstand the Safety Earthquake Event (SEE) forces until the shear keys are repaired. The zone of the SFOBB considered is the load transfer mechanism at Pier E2 from the superstructure bearing and shear key supports to the tops of the columns.

In the current context, a peer review involves determining if the work being reviewed meets the overall project safety requirements. It is not necessary to agree completely in a quantitative sense so long as the conclusion of the review is adequately supported.

Based on the engineering studies described herein, MM concludes that the concept of temporarily shimming the bearings at Pier E2 of the San Francisco-Oakland Bay Bridge as described in the July 15th information package entitled "Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S1land S2", and the proposed details, will provide more than sufficient capacity between the superstructure and the strut at Pier E2 to resist the design Safety Evaluation Earthquake. The service and seismic design forces in Pier E2 are not changed significantly by the redirection of seismic forces resulting from the shimming. Therefore the response of the concrete strut and columns are not expected to be different. Other than the possibility of scratching the paint system either when the shims are inserted or as the bridge rotates in service, there does not appear to be any reasonable possibility of damaging the bridge as a result of installing and removing the shims. Assuming that the rest of the structure has been properly designed, we conclude that the safety of the traveling public is improved by moving traffic on to the new bridge.

The exact distribution of forces in the bearings and shear keys is highly dependent on the planned and accidental tolerances (gaps) between various components. A precise quantification of the forces is, for practical purposes, unknowable. It is, however, possible to make estimates of the reasonable range of possibilities rather than relying on a single solution; MM’s assessment is based on that approach. Various assumptions have been used to place reliance on the set of shear keys and on the set of shimmed bearings, separately and in combination. Analysis of the resulting distribution of forces was the basis for concluding that the temporary shimmed condition provides sufficient resistance to meet the demands from the SEE.

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1. Authorization

Engineering services required to develop this report were authorized by the Bay Area Toll Authority by Agreement of July 16th, 2013.

2. Scope

The following technical Scope is excerpted from the Agreement of July 16th:

The services to be performed by Consultant shall consist of services requested by the Project Manager or a designated representative including, but not limited to, the following:

Task 1:

Consultant shall provide an independent review of the engineering analysis and strategy as related to the proposal to shim the existing bearings at Pier E2 to achieve the seismic design requirements of the new east span of the San Francisco-Oakland Bay Bridge.

Consultant shall conduct a peer review of the suitability of the "Shim Concept" to provide sufficient seismic capacity at Pier E2 of the San Francisco-Oakland Bay Bridge to withstand the Safety Earthquake Event (SEE) forces until the shear keys are repaired. In performing the work, Consultant shall use:

1. *Load demands at Pier E2 from the analysis of the SEE pushover by the engineer of record (EOR) as summarized in "Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S l and S 2" dated July 15, 2013* (July 15th information package)

2. *Details of the bearings, shear keys and cap beam at Pier E2 and the supports of shear keys and bearings in the orthotropic box girders provided by EOR; and*

3. *Other calculations, plan sheets, shop drawings and engineering summaries prepared or supplied by request to the EOR as may be needed.*

The zone of the SFOBB to be considered is the load at Pier E2 from the box girder bearing and shear key supports to the tops of the columns.

The following is expressly not included:

1. *Review of the failures of the 2008 anchor bolts;*
2. *Independent seismic analysis or evaluation other than force transfer in the zone identified above;*
3. *Any capacity of the box girders or the substructure other than the force transfer zone*

identified above;

4. *Any evaluation of the repair of the shear keys (S l and S2) or the resulting suitability;*
5. *Any evaluation of the original design of the cap beam at Pier E2 other than as may be needed to evaluate the changes to the load path caused by the temporary shimming; and*
6. *Any other function of the shear keys and bearings other than transferring seismic demands in the temporary shimmed condition.*

Deliverable:

Independent Review Report

3. Background

Figure 1 through Figure 9, from the July 15th information package, were provided by the EOR, TYLin International.

3.1 Seismic Forces and Assumed Behavior of Bearings and Shear Keys

The original load path with clearances preventing the bearings from participating in resisting horizontal and transverse shear is illustrated in Figure 1. The four bearings were intended to carry only vertical loads, the four shear keys were intended to carry transverse shear, and shear keys 1 and 2 were intended to carry longitudinal shear.

Shimming is proposed to close the gaps and provide a contact load path through the bearing to transmit longitudinal and transverse shear. The location of the shims is shown in Figure 2. Due to failure of many of the anchor bolts at shear keys S1 and S2, it has been decided that all shear capacity at these shear keys should be ignored until they are repaired. With the shims in place, the proposed load path is shown in Figure 3. The bearings carry vertical loads as in the original load path, but are now intended to carry the entire longitudinal shear and a portion of the transverse shear. Shear keys 3 and 4 are still intended to carry some of the transverse shear, but the distribution of shear between the bearings and shear keys requires consideration. With shear keys S1 and S2 considered ineffective due to anchor bolt fractures, the longitudinal shimming is required in any event to provide an effective load path in that direction until the permanent repair of shear keys S1 and S2 in completed.

The Safety Evaluation Earthquake (SEE) demands and the nominal shear capacities of the bearings and shear keys for three scenarios (load paths) are shown in Figure 4; load path "A" is as designed, load path "C" is based on the shimming of the bearings and the participation of shear keys 3 and 4, i.e. the starting point for this investigation.

The magnitudes and directions of seismic forces given in the July 15th information package, excerpts of which are repeated below, are accepted without review per scope:

Bearing forces were extracted from a seismic (time history) analysis of the self-anchored suspension bridge including the bearings and shear keys. The total longitudinal, transverse, and vertical loads transferred from the westbound and eastbound box girders to Pier E2 were extracted from the analysis and distributed to the bearings and shear keys in accordance with the plans. The bearing loads are shown in Table 1.

Normal functioning of the bearing corresponds to the case "Shear Key Engaged". The bearing is only required to carry vertical loads. These are either downwards - case C - or upwards - case U. Upwards loads are of greatest concern and are addressed in this report. A "safety factor" of 1.4 is applied to the calculated loads from the seismic analysis.

The bearing is also intended to function as a secondary mechanism to resist longitudinal and transverse loads should the shear keys fail. The three cases of greatest interest are those corresponding to the peak uplift on the bearing (case U), the peak transverse load (case T), and the peak longitudinal load (case L). In each case the orthogonal loads occurring simultaneously with the peak loads are also tabulated (and analyzed). These loads are applied with a "safety factor" of 1.0, since they are based on the conservative assumption that the shear key has failed.

Bearing Forces (SF = 1.4)

Case	*Case*	*Trans.*	*Long.*	*Vert.*
Shear Key Engaged	*C*	*0*	*310*	*81104*
(Load Path A)	***U***	***0***	***108***	***-13355***

Bearing Forces (SF = 1.0)

Case	*Case*	*Trans.*	*Long.*	*Vert.*
	C	*10799*	*4770*	*57932*
Shear Key Failed	***U***	***25287***	***1628***	***-9539***
(Load Path B&C - See	***T***	***30496***	***8186***	***16441***
Note)	***L***	***1340***	***13232***	***19255***

Note: The same seismic demands are conservatively assumed for Load Path C.

Table 1, Bearing Loads

As mentioned previously, the loading on the model is assigned at the CG of the bearing shaft, which transfers the focus from the bearing upper housing to the bearing lower housing.

The load is modeled as pressure loading applied at relevant surfaces, with some simplifications.

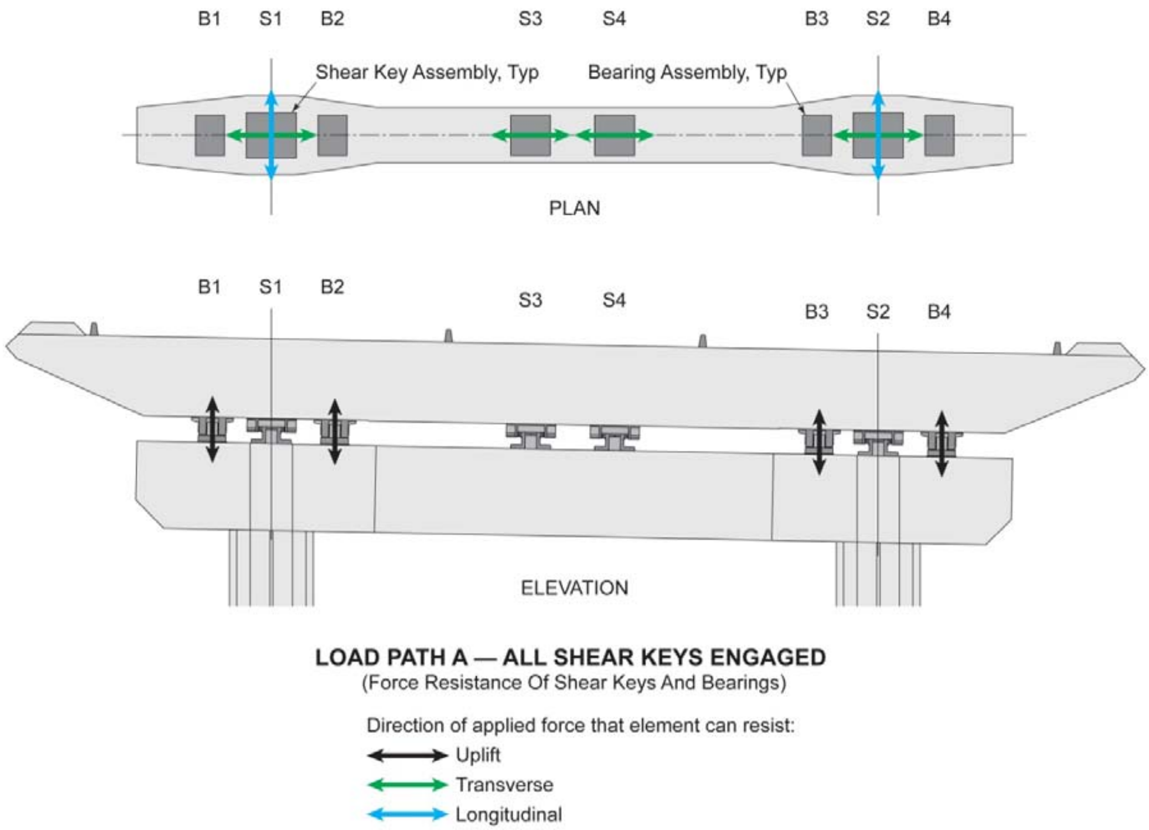


Figure 1 - Original load path through shear keys and bearings

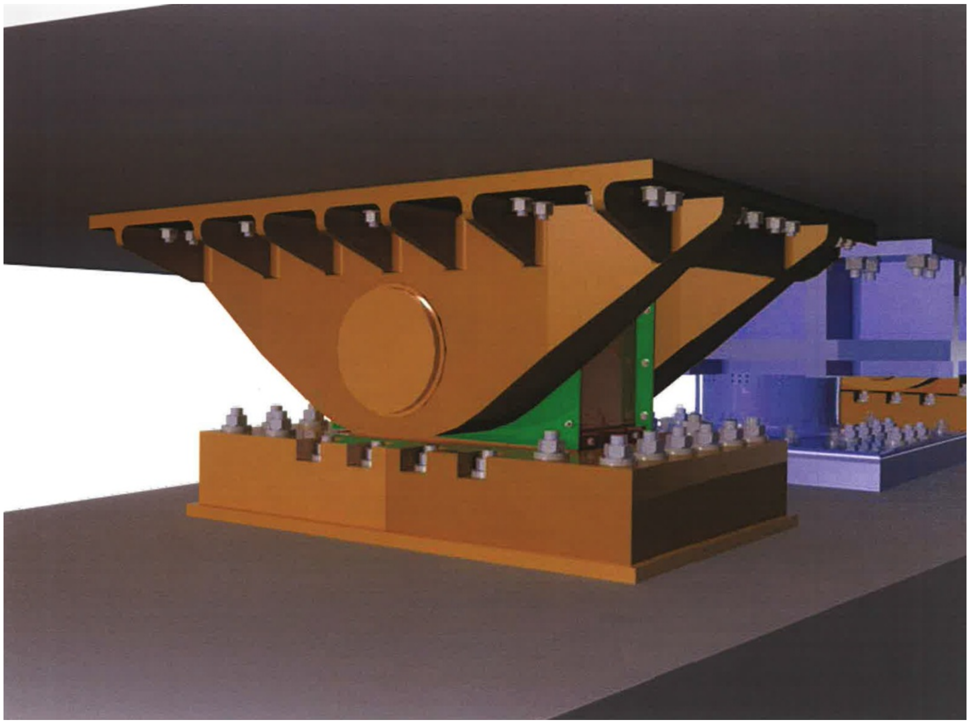


Figure 2 - Shimmed bearing

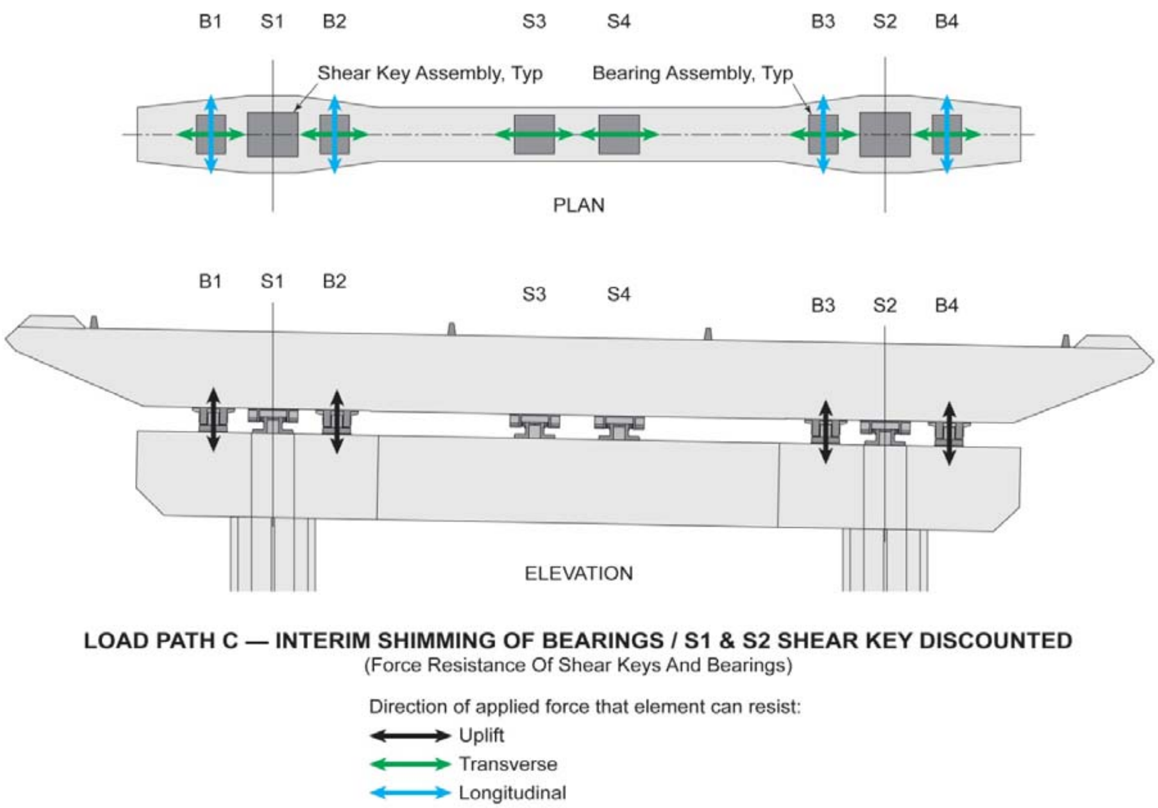
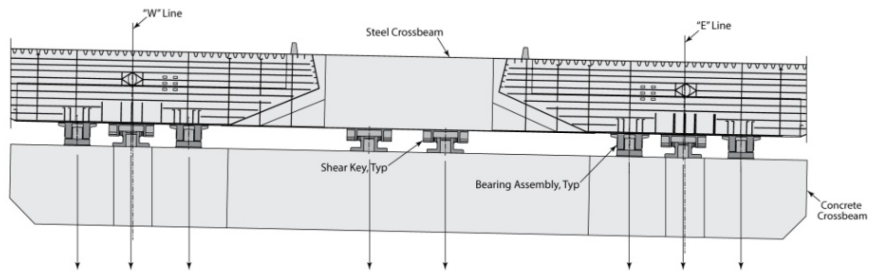


Figure 3 - Revised load path through shear keys and bearings



		Direction	B1	S1	B2	S3	S4	B3	S2	B4	TOTAL (MN)	SAFETY FACTOR
DEMAND	SEISMIC DEMAND FOR SEISMIC SAFETY EVALUATION (SEE) EARHQUAKE	Long									50	
		Trans									120	
CAPACITY	LOAD PATH A: (ALL SHEAR KEYS ENGAGED)	Long	0[a]	42	0[a]	0[c]	0[c]	0[a]	42	0[a]	84	168%
		Trans	0[b]	42	0[b]	42	42	0[b]	42	0[b]	168	140%
	LOAD PATH B: (ALL SHEAR KEYS DISCOUNTED) (BEARINGS ENGANGED)	Long	15	0	15	0	0	15	0	15	60	120%
		Trans	30	0	30	0	0	30	0	30	120	100%
	LOAD PATH C: (INTERIM SHIM OF BEARINGS) (S1 & S2 SHEAR KEYS DISCOUNTED)	Long	15	0	15	0[c]	0[c]	15	0	15	60	120%
		Trans	30	0	30	42	42	30	0	30	204	170%

a. 30 mm gap in the longitudinal direction. Bearing (B1, B2, B3, and B4) engage after 30 mm gap is closed by displacement.
b. 20 mm gap in the transverse direction. Bearing (B1, B2, B3, and B4) engage after 20 mm gap is closed by displacement.
c. 43 mm gap filled with neoprene open cell. Shear Keys (S3 and S4) engage in the longitudinal direction after 43 mm gap is closed by displacement.

Figure 4 - Reported demand and capacities of shear keys and bearings

3.2 Material Properties (Provided by EOR, anchor bolt capacity confirmed by MM)

- Concrete strength: f'_c = 55.2 MPa (8 ksi)
- Reinforcing steel: F_y = 414 MPa (60 ksi)
- Structural steel plates: F_y = 354 MPa (50 ksi)
- Steel castings: (354 MPa, 550 MPa (50 ksi, 80 ksi)
- Anchor rods were greased and taped:
  - Tensile area = 5.97 sq in
  - Force @ F_u=140 ksi for 3" bolt = 835.8K = 3.718 MN tensile strength
  - Stressed to 0.75: tensile force = 2.789 MN per bolt
  - Assuming 10% loss: 2.510 MN per bolt

- Prestressing strand: F_u = 1,862 MPa (270 ksi)

3.3 Previously Stated Shear Capacities of Bearings and Shear Keys

The bearings have split base plates such that the longitudinal capacity is one-half of the transverse capacity. The steel-steel coefficient of friction was assumed to be 0.50 yielding a sliding capacity 1.25 MN per bolt. This extends to 15.0 MN for 12 bolts in the longitudinal direction and 30.0 MN for 24 bolts in the transverse direction. (See possible reduction due to load path through hold down assembly discussed below.)

For the shear keys, the nominal capacity provided by the EOR was 42 MN based on first yielding indicated by a finite element analysis. For shear keys 3 and 4, the sliding capacity was determined to be 81 MN using a steel-concrete coefficient of friction of 0.67 and 48 bolts. This is much greater than the first yield capacity.

4. Approach

4.1 General

As indicated in the Scope, this investigation is focused on the force transfer zone between the bottom of the orthotropic box girders (OBGs) and the Pier E2 cross girder (CG) to the interface of the strut and columns. All structural issues related to the change in load path resulting from shimming the bearings and discounting shear keys 1 and 2 are believed to be concentrated in this area which is comprised of the pier strut including the contact surface and load transfer between the base plates and the concrete, the bearings and shear keys, and the capacity of the OBGs and cross girder to transfer loads to the bearings and shear keys.

The following areas were investigated relative to the revised loads resulting from the shimmed bearing concept:

1-Analysis of load transfer in bearings and shear keys including the capacity of friction interfaces, a first level approximate analysis of changes in clamping forces in bearings, and a more refined analysis of the effects of gaps and nonlinearities on load sharing among shear keys and bearings.

2-Capacity of OBGs and the Pier E2 cross girder to deliver load to the shimmed bearings and shear keys, respectively.

3-Capacity of the pier strut.

4-Transfer of horizontal load from bearing base plate to strut to column.

5-Consideration of potential to damage the permanent bearings or other components if shims are installed.

The exact distribution of forces in the bearings and shear keys is highly dependent on the planned and accidental tolerances (gaps) between various components and is, for practical purposes, unknowable. However, it is possible to make estimates of the reasonable range of possibilities rather than placing reliance on a single solution; MM's assessment is based on that approach, sometimes referred to as bounding the solution. Various assumptions have been used to place reliance on the set of shear keys and on the set of shimmed bearings, separately and in combination. At the extreme ends of the bounds, it could be assumed that the shear keys alone carry the transverse component of the SEE, or that the bearings alone carry the load. Other acceptable joint participation possibilities within these bounds may also be found to provide sufficient capacity. The possibility of unequal distribution among the individual shear keys and individual bearings in the two extremes should be considered at least subjectively.

The difference between the individual forces provided in the longitudinal and transverse directions and a vector sum of those forces was found to be small, on the order of a few percent; the individual forces will often be used herein for simplicity.

4.2 Analysis of Load Transfer in Bearings and Shear Keys

4.2.1 Free Body Diagrams

Figure 5 through Figure 9 illustrate the basic flow of forces through the shear keys and the bearings. For both types of assemblies, the shear from the seismic event is transferred from the upper housing to a lower housing at discrete points, shown as the shear key bushing or the shaft in the case of the bearing. The rest of the load paths in Figure 5 through Figure 9 are global statics and will be used as a starting point for the investigations reported herein. The distribution of stress within the assemblies and the participation of individual anchor bolts can be more complex.

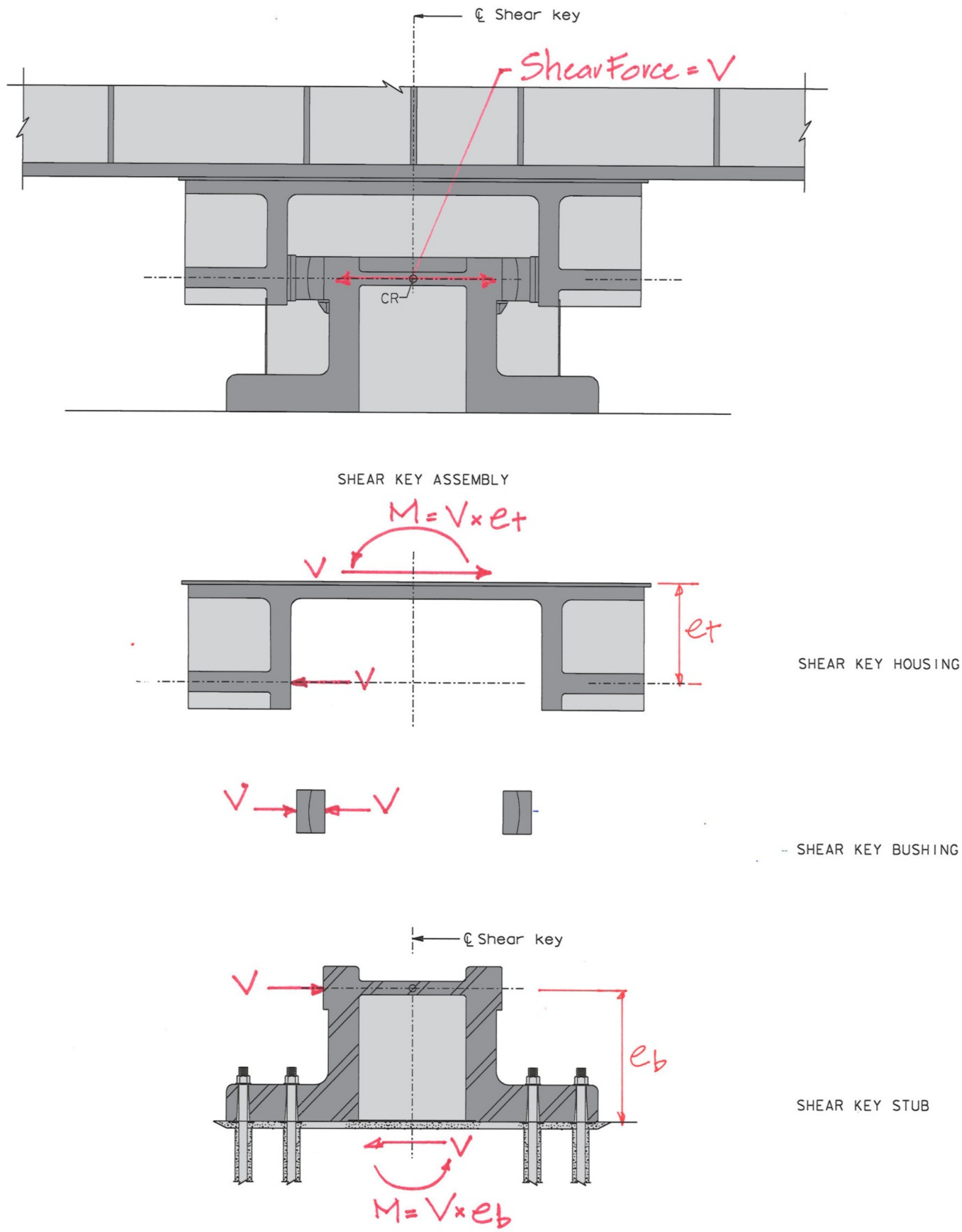


Figure 5 - Load path through shear key

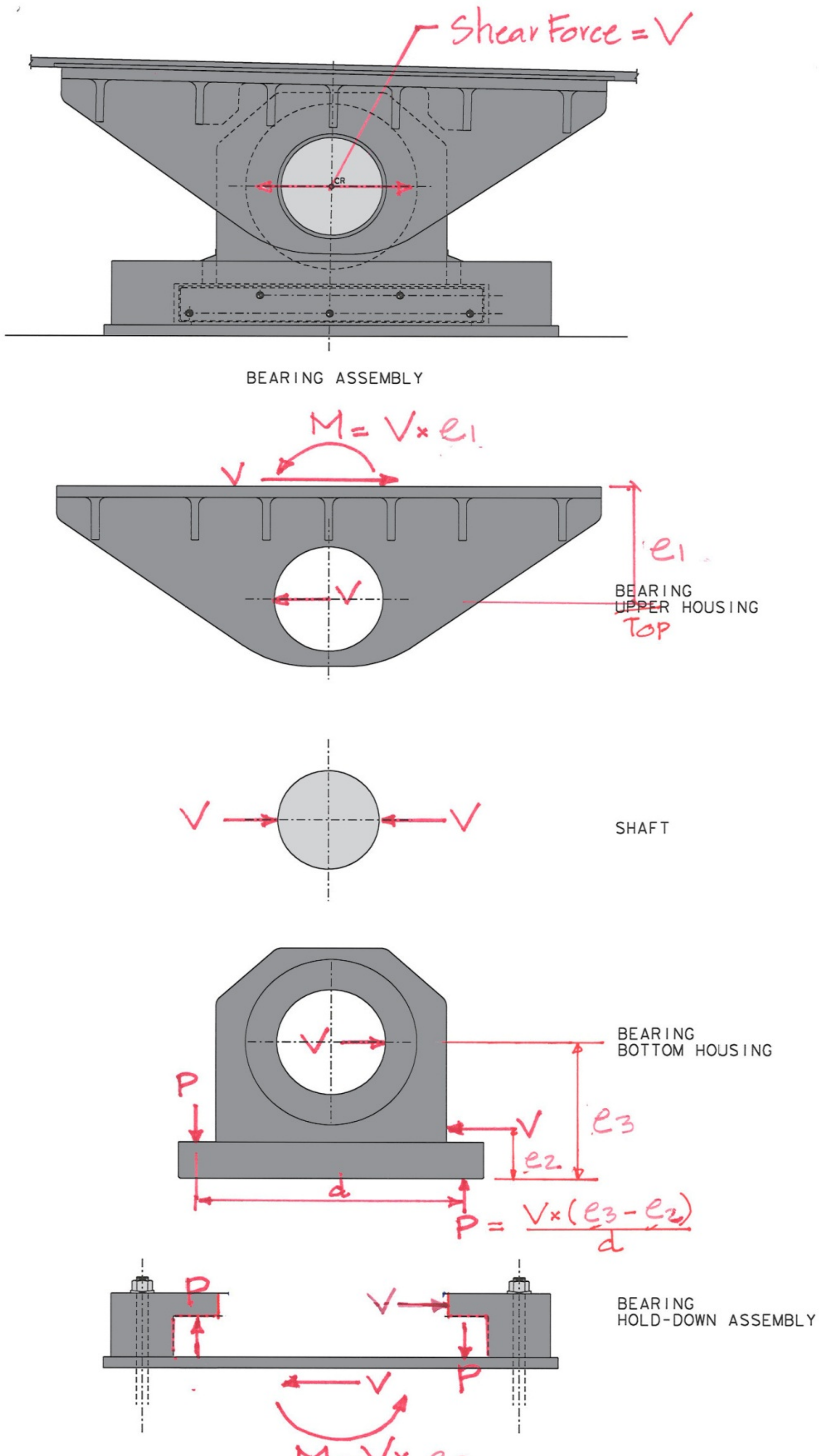


Figure 6 - Load path through bearing - longitudinal shear

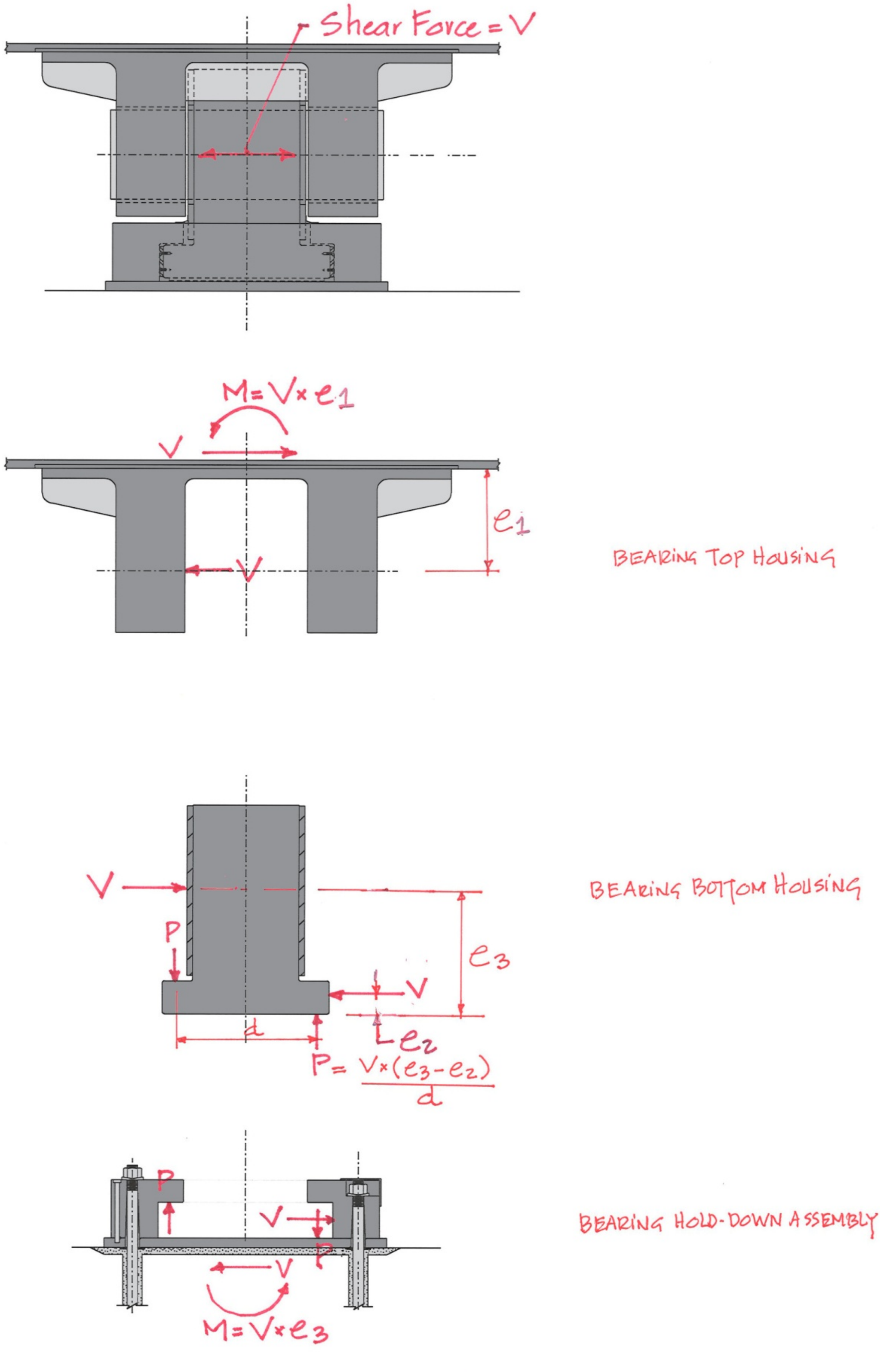


Figure 7 - Load path through bearing - transverse shear

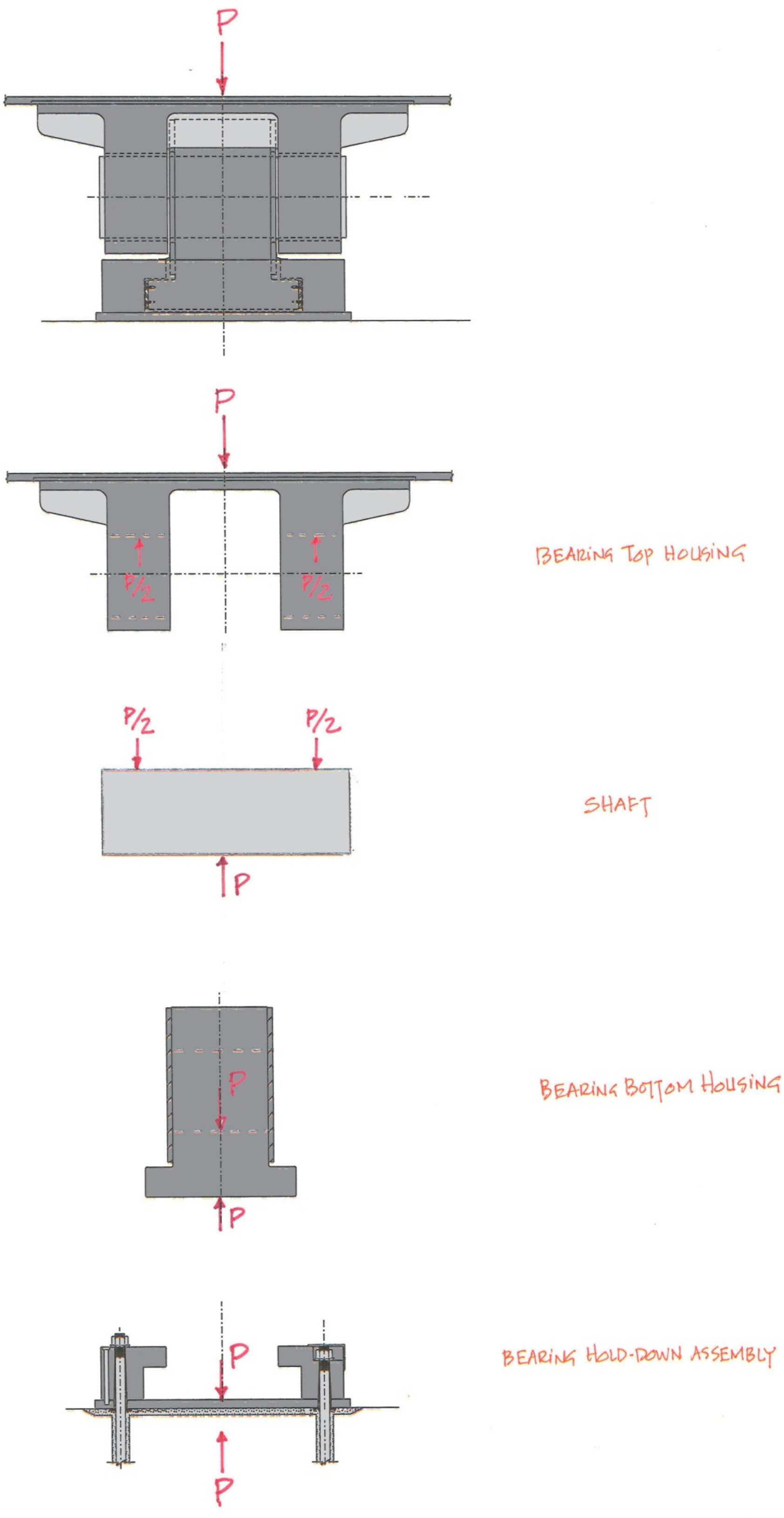


Figure 8 - Load path through bearing - compression

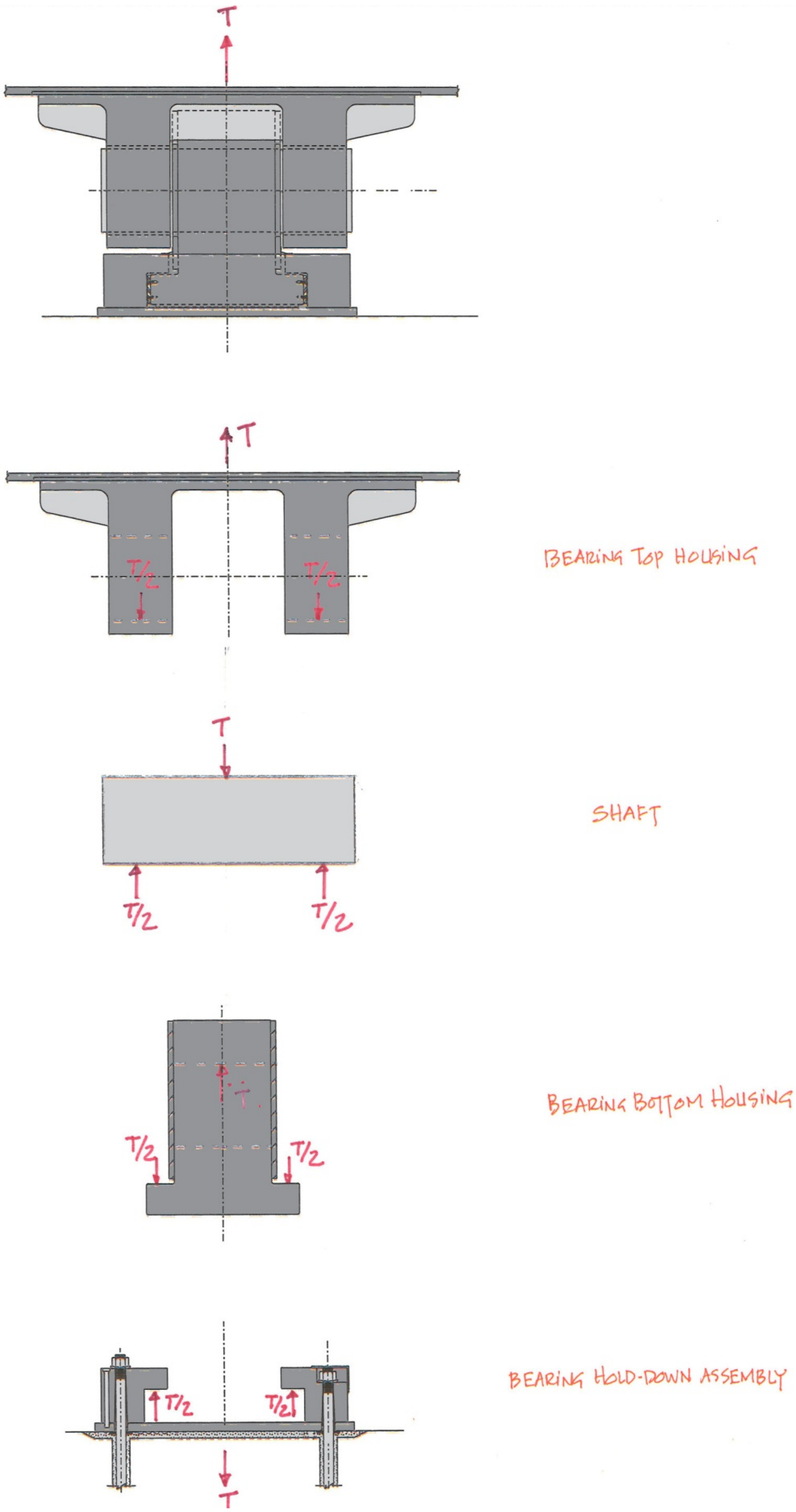


Figure 9 - Load path through bearing - uplift

4.2.2 Coefficients of Friction at Interfaces

The coefficient of friction of 0.50 used for the steel-steel interface in the bearings is consistent with the AASHTO Class B coating factor for slip-critical high strength bolts. EOR has confirmed orally that Class B surface preparation was required; written confirmation in specifications or shop drawings has been requested**.** Since the steel-steel coefficient of friction is less than the steel-concrete coefficient there is no need to check the latter capacity.

The coefficient of 0.67 used for steel anchored to concrete in the shear keys is less than (more conservative than) 0.70 used by AASHTO for shear friction. The bottoms of the base plates were machined (roughened) with 5 mm deep recesses as shown on shop drawings provided by the EOR further improving the performance. Friction capacity using the 0.67 factor results in about 81 MN or about twice the nominal capacity of 42 MN provided in the July 15th information package. The corresponding capacity of the steel to steel interface at the upper housing is approximately 100 MN.

4.2.3 Capacity of Bearings and Shear Keys (individual and combined) at 100% SEE – Approximate Mechanics Analysis

First yield capacity for the shear key stub is about 76 MN for bending alone and 73 MN in shear alone based on simple mechanics and a yield strength of 345 MPa (50 ksi). Conservatively using the maximum shear and bending stresses as though they occurred at the same point on the stub, results in a von Mises capacity of 53.3 MN at first yield. MM's finite element analysis of the shear key stubs indicates first yield at 49 MN. The EOR reports that first yield load of the stub based on FEA is 42 MN and that value will be used in this section.

Referring to the bottom panel of Figure 7, the load path in the bearings causes compression due to overturning from the transverse shear to go directly into base plate through a lubricated surface so as to bypass the hold down assembly while the tensile component goes directly into the tang on the hold down assembly resulting in a net reduction of clamping force on the steel-steel interface. As shown in the bottom panel of Figure 9, uplift goes into the hold down tang which also reduces clamping although this is partially offset on one side by overturning compression. (The overturning effect is much larger than the axial effect). The net result is a reduction in clamping force and an associated reduction in shear capacity compared to that indicated by the estimates in the July 15th information package (bearings: 30 MN transverse and 15 MN longitudinal). Results are summarized below and more details are given in Appendix 1.

For the load case with the maximum transverse force, the shear capacity of the bearings is currently estimated, based on rough hand calculations, to be about 25.7 MN or about 86% of the nominal 30 MN capacity. The capacity of the 4 bearings alone would be 103 MN, less than the SEE demand of 120 MN, so some participation from the shear keys is needed to resist the SEE. Using the concept of bounding the possible solutions, consider that all of the shear keys and bearings engage equally and are limited to 86% of their nominal total shear capacity (42 MN and 30 MN, respectively) to protect the bearings. The transverse resistance would be about 175MN, greater than the 120 MN needed for the maximum transverse force case. Alternatively, if the shear keys reached their nominal 42 MN capacity and the bearings were limited to their shear capacity based on the reduced clamping force (25.7 MN), the total resistance would be 187 MN.

In the maximum uplift case, the shear capacity of the bearings is further reduced to 21.7 MN or 72% of their capacity based on the unreduced clamping force for a total capacity of the 4 bearings of about 87 MN which is less than the transverse shear associated with the maximum uplift, estimated by proportion to be about 100 MN . Again, some participation from the shear keys is needed. Following the logic used above, if the total shear resistance was based on all bearings and shear keys being limited to 70% of their nominal capacity to protect the bearings, the resistance would be about 143 MN. This is greater than the 120 MN maximum transverse shear and even greater than the proportioned 100 MN transverse shear for the uplift case. If the shear keys reached their full resistance and the bearings reached their reduced capacity (21.7 MN), the shear resistance would be 171 MN.

As shown in the Section 4.2.4, in which the displacement compatible system behavior of the shear keys and bearings is discussed, MM's analysis shows that, for a range of assumed gaps, more of the transverse load is carried by the shear keys and less by the bearings than consideration of their nominal capacities would indicate. The net effect is that the displacement compatible demands on the bearings are smaller than the clamping reduced capacities presented in this section.

Further bounding the analyses, if full reliance were placed equally on the two shear keys each would have to carry 60 MN. This is greater than the nominal 42 MN capacity but less than their sliding capacity of about 81 MN. This implies that either some yielding of the shear key stub or housing, or participation of the bearings, would be necessary to carry the full load. This is investigated using more refined analysis in the next section.

Longitudinal shear in bearings is not affected by the reduced clamping force issue discussed above because the reduced clamping would be on the side of the split base plate which is neglected in calculating the shear capacity. Thus the 15 MN resistance per bearing or 60 MN total for 4 bearings given in Figure 4 is deemed appropriate and more than the 50 MN reported to be required by the SEE.

The shear keys are not affected by the reduced clamping force issue because tension and compression, from overturning, are both carried by the same elements so there is no net change in the clamping force.

4.2.4 Capacity of Bearings and Shear Keys (individual and combined) at 100% SEE – Refined Finite Element Analysis

A potential issue that was identified early in the review is the sharing of load between the shear keys and the bearings. Both the shear keys and the bearings, when fully engaged, provide a very stiff load path for the seismic forces between the OBG and Pier E2. As noted earlier, the bearings were designed with sufficient clearances to prevent them from transferring horizontal load. Some, but not all of these clearances will be addressed by the proposed shimming operations. In the transverse direction, there is a horizontal gap with an as-designed value of 2 mm, between the lower housing and the hold down assembly, which will remain after shimming. See Figure 10. Additionally, there is an as-designed 3mm vertical gap that allows the lower housing to rotate a small amount. These gaps allow a certain amount of free movement of the bearings before they become engaged in transferring lateral forces which will affect the distribution of loads between the shear keys and the bearings.

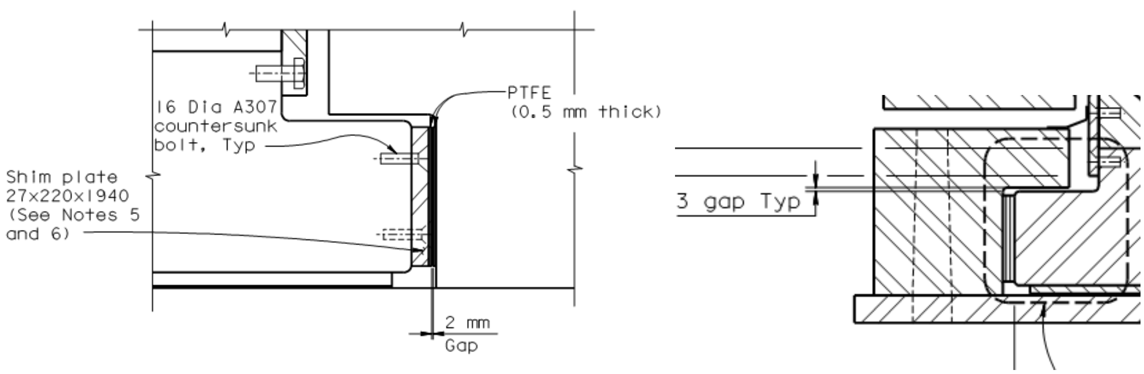


Figure 10 - Details from the design drawings showing 2 mm and 3 mm gaps

Two analyses were performed to evaluate the distribution of load between the shear keys and the bearings. A 3D FEA including the hold down assembly and the lower housing was performed to determine the force-displacement behavior in the as-designed condition. The model included the 2mm and 3mm gaps, as well as compression-only contact surfaces. Figure 11 shows a general view of the model and Figure 12 shows the deformed shape with stress contours. A separate FEA was performed on the top housing and its stiffness was included to find the total bearing force-displacement relationship. Additionally, the force-displacement relationship was calculated by hand using typical engineering approximations.

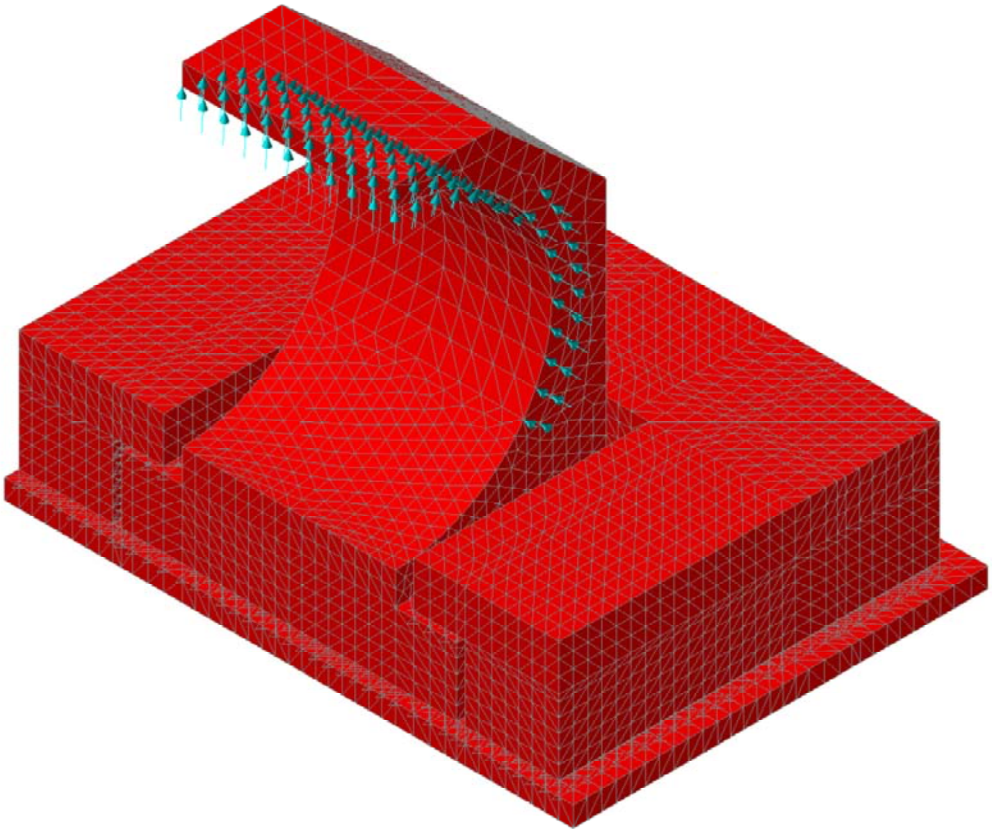


Figure 11 - Finite element model of the lower housing and hold down assembly

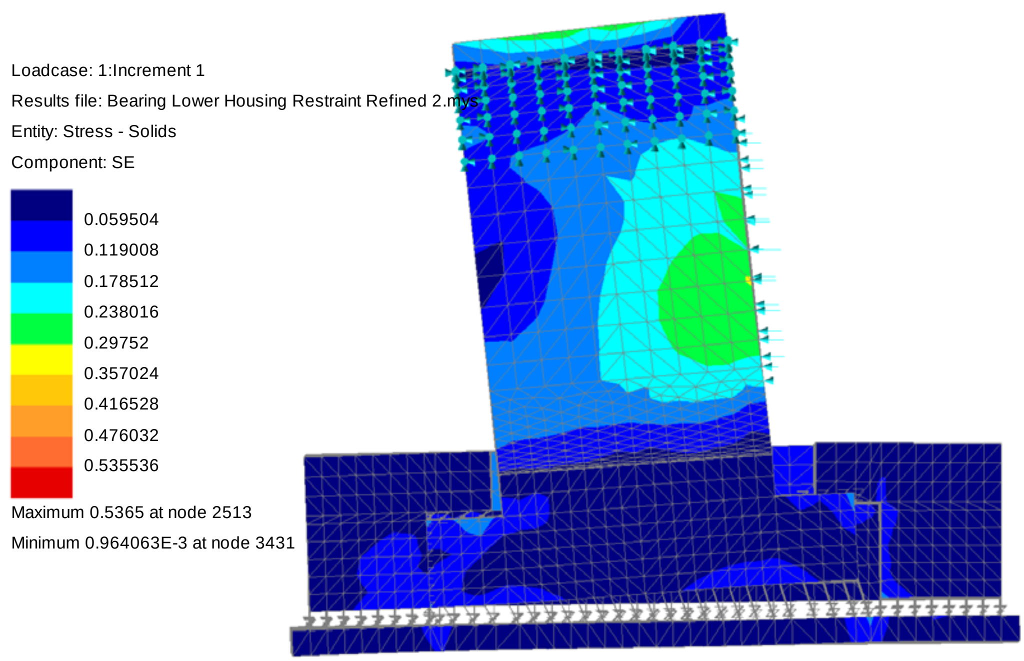


Figure 12 - End-on view of the deformed shaped under lateral and uplift loading

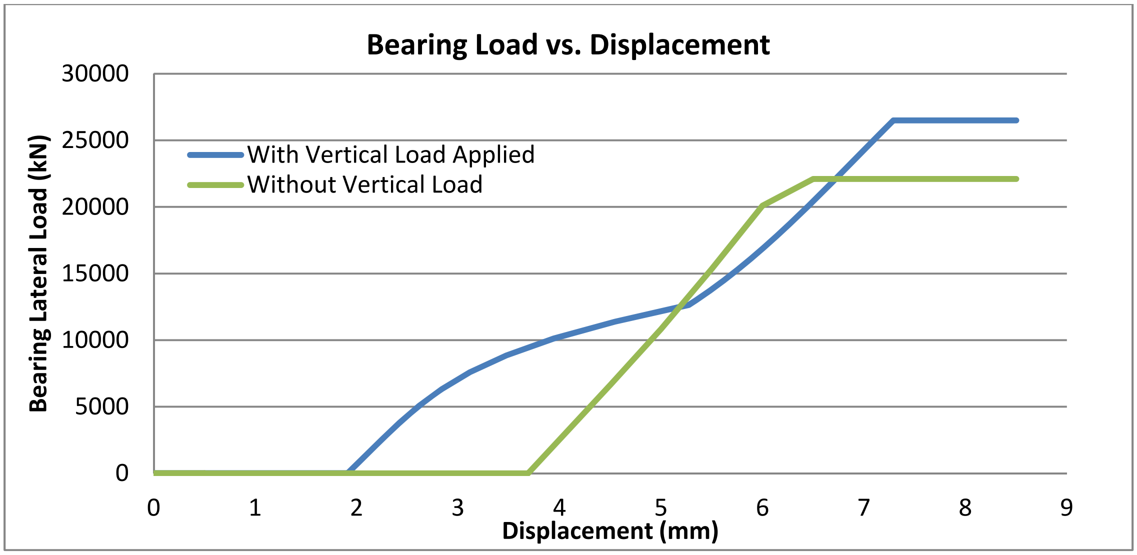


Figure 13 - Load vs. displacement for bearings with varying vertical load

Figure 13 shows the lateral load vs. displacement relationship for two conditions; with vertical load (uplift in the case shown) and with no vertical loads. There is a coupling between lateral displacement and vertical load through the rotation of the lower housing. Because the presence of vertical load during the times of large lateral loading cannot be guaranteed, the curve without vertical load was used in further evaluations. The plateaus in the graphs are the approximate load levels at which friction is overcome, and the bearings begin to slide on the steel-to-steel interface.

Finite element models were also developed for the shear key; both the stub and the housing were modeled in separate analyses. Because of the potential for exceeding the stated capacity of 42 MN for the shear keys, nonlinear material behavior was included in the models. The lateral stiffness of the shear keys was determined from these analyses, as well as from independent hand calculations. Figure 14 shows the load vs. displacement relationship for the shear key. The curve is quite linear to loads well in excess of the nominal capacity, with softening becoming apparent only at very large load levels (>70 MN). A 0.5 mm gap was assumed between the stub and the spherical ring. It was indicated that the actual free play in the shear keys may be larger and this is explored below.

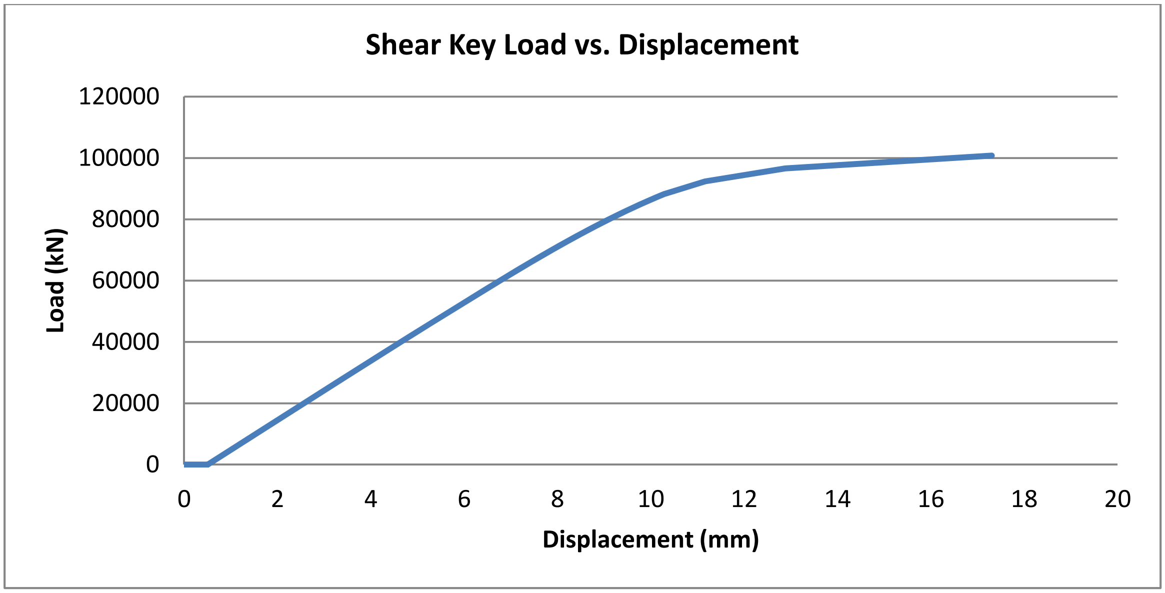


Figure 14 - Shear key load vs. displacement

A total force vs. displacement curve can now be assembled from the individual curves for the interface between the OBG and Pier E2 with 4 bearings and 2 shear keys. This is shown in Figure 15. Table 2 lists the forces in the shear keys and bearings for several displacements based on the curves above. Because the bearings do not carry load until the various gaps have closed, the shear keys carry a majority of the seismic load.

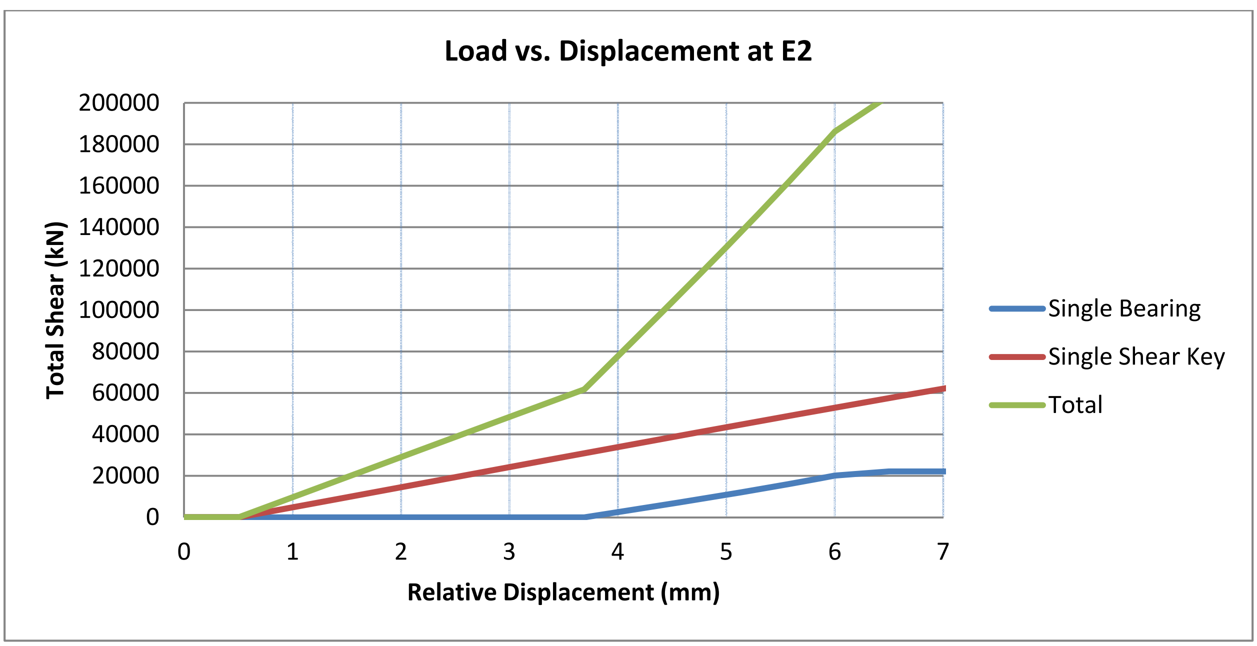


Figure 15 - Combined load vs. displacement for bearings and shear keys

Table 2 - Loads in bearings and shear keys at various displacements

Displ. (mm)	Load (kN)		
	Ea. Bearing	Ea. Shear Key	Total
3.69	0	30883	61767
4.8057	9206	41589	120001
4.8487	9566	42000	122264
5.6799	17059	49882	168001
6.527	22100	57803	204006

Due to the very large stiffness of the force transfer mechanisms at play, very small variations in the size of gaps can have a significant effect on the distribution of load between the components. The previous analyses can be repeated with various assumed gaps in both the shear keys and bearings. Two cases were examined: the gaps in the shear keys and the shimmed bearings are approximately equal and a relatively greater gap in the bearings such that they do not become engaged until approximately 6 mm relative displacement. These are shown in Figure 16. Table 3 and Table 4 list the forces in the components at various displacement levels for these two cases.

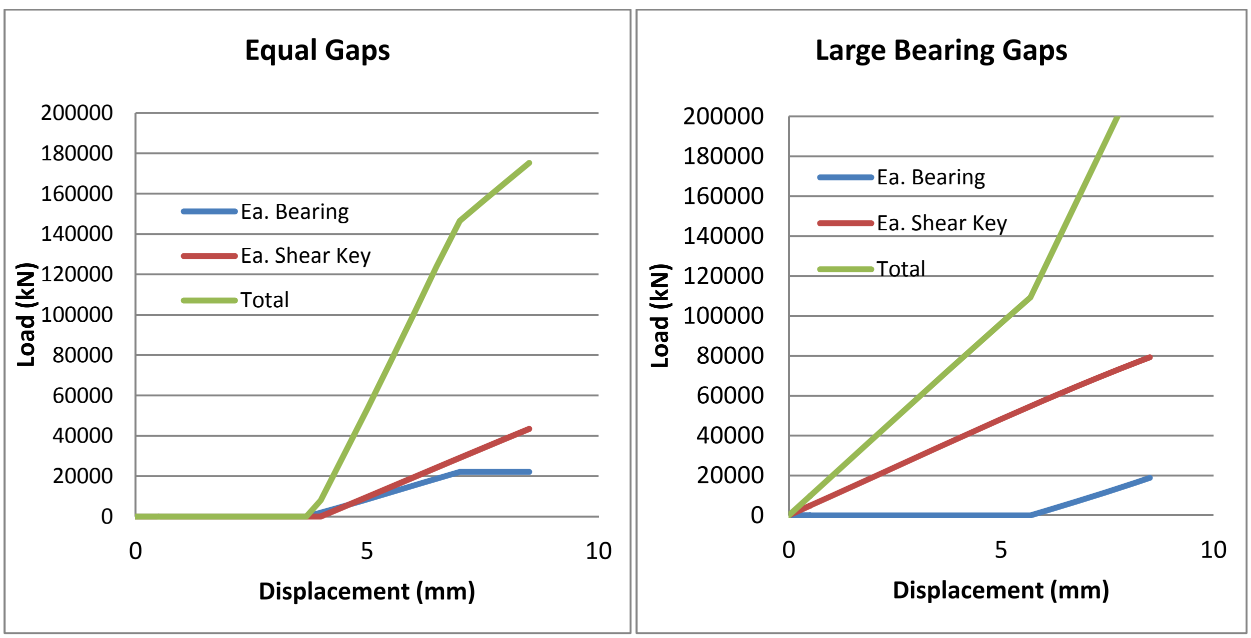


Figure 16 - Load vs. displacement for variable relative gaps

Table 3 - Loads in components with equal gaps in bearings and shear keys

Displ. (mm)	Load (kN)		
	Ea. Bearing	Ea. Shear Key	Total
3.69	0	0	0
4	1998	0	7990
6.4194	18279	23443	120001
6.939	22100	28464	145326
8.1182	22100	39800	168000

Table 4 - Loads in components with 6 mm relative gap between shear key and bearings

Displ. (mm)	Load (MN)		
	Ea. Bearing	Ea. Shear Key	Total
4.3481	0	42000	84000
5.9301	1547	56906	120000
7.0206	8579	66835	168000

Based on the above investigations, the best approach to evaluating the distribution of loads between the shear keys and the bearings is to bound the solution. In addition to the uncertainty in the magnitude of the free play between bearings and shear keys, it is likely that variations exist between the two shear keys and among the four bearings. This would increase the load any one component may experience beyond that found assuming equal distribution among the same components. Another factor is the effect of dynamic impact. During a seismic event, the gaps in these components will close at some non-zero velocity, and this will result in localized impact loading. Simple 2 DOF studies were made to determine the order of magnitude of these impact forces; they were found to be very sensitive to assumed damping levels. Regardless, impact values of 10% or more do not seem impractically large. Therefore, the range of loading to evaluate the shear keys should be from 23.4 MN to 56.9 MN, and for the shimmed bearings from 1.5 MN to 18.3 MN. Based on the evaluation of the capacities of the bearings, there remains approximately 18% reserve capacity before sliding. For the shear keys, although yielding was found to occur at a load in the vicinity of 50 MN, significant capacity exists above that load. Based on the nonlinear finite element analysis discussed in Appendix 2, a maximum capacity of 65 MN appears to be reasonable. This load level limits the amount of inelastic behavior while still taking advantage of the large reserve capacity of the shear keys. Based on this, the reserve capacity of the shear keys is approximately 15% over the SEE demand. Thus, both the bearings and the shear keys have adequate reserve to cover uncertainties in the magnitudes of the loads, including some unequal load sharing within set of like units.

A question might be asked as to why shim the bearings if shear keys S3 and S4 appear to have adequate capacity to carry the full SEE demand by themselves. As discussed above, there are multiple sources of uncertainty regarding the magnitude of the loads and the behavior and robustness of the system is improved by the addition of the transverse shimming.

4.3 Capacity of OBGs and the Pier E2 Cross Girder to Deliver Load to the Shimmed Bearings and Shear Keys, Respectively

4.3.1 Introduction

The steel orthotropic box girder (OBG) and steel cross girder (CG) were evaluated at Pier E2 for the seismic forces applied through the bearings and the S3 and S4 shear keys prior to the completion of the S1 and S2 shear keys. The bearing and shear key locations were examined independently to determine capacities. In some cases, inconsistent conservative assumptions were made regarding loads or load paths. For example, the tension in anchor bolts might be maximized to analyze bearing plates and stiffeners, and then minimized to determine friction from clamping. Another example would be that alternative load paths would be ignored when determining demand in a given component. The demands to determine demand to capacity ratios (D/C) were assumed to be as listed in Table 5. Note that in keeping with the approach to bound possibilities, it has been assumed that either the shear keys or the bearings carry all of the transverse force from the design SEE. For the purpose of evaluating the OBGs and the CG at Pier E2, these loads are local to the individual element and are not additive, i.e. in the temporary shimmed condition the OBGs need only be evaluated for the loads from the bearings and the cross girder need only be evaluated for the loads from the shear keys.

Table 5 - Seismic demands (MN)

	B1	S1	B2	S3	S4	B3	S2	B4
Longitudinal	15	0	15	0	0	15	0	15
Transverse	30	0	30	60	60	30	0	30

4.3.2 Bearings

There are 4 bearings at Pier E2, two under each box girder, as illustrated in Figure 4. The OBG is attached to the bearings using 56 – 2" diameter A354 Grade BD rods anchored into stiffened anchor seats within the OBG. These anchor rods are pretensioned to clamp the bearing onto the thickened bottom flange (key plate) of the OBG. The resulting friction carries the longitudinal and horizontal forces between the OBG and the bearings. The various elements including longitudinal shear plates, transverse webs, and various stiffeners and diaphragms of the OBG at the bearing attachment are illustrated in the design drawings in Appendix 3C.

The stiffened anchor seats were analyzed for the initial tensioning force, as well as the entire local OBG assemblage being analyzed for the transfer of the seismic forces between the OBG and the bearing. The plates are all stiffened compact elements that are assumed to be able to reach their yield point of 345 MPa (50 ksi). Table 6 lists the demand to capacity ratios (D/C) for various elements. The calculations for these values are contained in Appendix 3A. Similar calculations performed by the EOR with comparable results are contained in Appendix 3B.

Table 6 - Demand/Capacity ratios OBG

Component	Force	Demand/Capacity – D/C (%)
1. Key plate	Longitudinal	15%
	Transverse	26%
2. Longitudinal shear plate	Longitudinal	72%
3. Floorbeam webs	Transverse	49%
4. Anchor bolt bearing plate	Bending	66%
5. Center brg stiffener plate	Axial	84%
	Shear	48%
6. Side brg stiffener plate	Axial	35%
	Shear	21%
7. Floorbeam web	Anchor assembly tearout	53%
8. 50mm x 700mm plates	Shear	95%
9. Bearing surface	Friction	95%
10. Local vertical OBG elements	Normal stresses	72%

It should be noted that the bearing surface sliding friction D/C ratio is not for the maximum seismic forces occurring simultaneously in all three orthogonal directions, but is for the maximum uplift of 9.5 MN along with the concurrent transverse and horizontal forces (see calculations in Appendix 3A).

The D/C ratios in Table 6 indicate that OBGs have more than sufficient capacity to resist the demands of the design SEE at pier E2. This conclusion is further supported by the analyses in Sections 4.2.3 and 4.2.4 which indicate that the bearings will not be subjected to the nominal 30 MN under the design SEE.

<u>4.3.3 Shear Keys 3 and 4</u>

Shear keys 3 and 4 are located 2900 mm either side of midspan of the steel cross girder at Pier E2 (see Figure 4 and Design Drawings in Appendix 3C). The CG is attached to the shear keys using 48 – 3" diameter A354 Grade BD rods anchored into stiffened anchor seats and 32 – 3" diameter anchor rods bolting the top plate of the shear key directly to the bottom flange of the CG. These anchor rods are pretensioned to clamp the shear key onto the thickened bottom flange (key plate) of the CG. The resulting friction carries the transverse seismic forces from the CG into the shear key. The various elements of the CG at the shear key attachment including floorbeam webs, diaphragms, and stiffeners are illustrated in the Design Drawings in Appendix 3C.

The stiffened anchor seats were analyzed for the initial tensioning force, as well as the entire local CG assemblage being analyzed for the transfer of the seismic forces between the CG and the shear key. The plates are all stiffened compact elements that are assumed to be able to reach their yield point of 345 MPa (50 ksi). Table 7 lists the demand to capacity ratios (D/C) for various elements. The calculations for these values are contained in Appendix 3A.

Table 7 - Demand/Capacity ratios steel cross girder

Component	Force	Demand/Capacity – D/C (%)
1. Key plate	Transverse	38%
2. Floorbeam webs	Transverse	74%
3. Anchor bolt bearing plates	Interface shear	80%
	Bending	65%
4. Center brg stiffener plate	Axial	81%
	Shear	92%
5. Side brg stiffener plate	Axial	42%
	Shear	37%
6. Diaphragms	Anchor assembly tearout	76%
7. Floorbeam web	Anchor assembly tearout	94%
8. Bearing surface	Friction	60%
9. Local vertical CG elements	Normal stresses	41%
10. Bending+axial global est.	Normal stresses	50%

The D/C ratios in Table 7 indicate that the CG at Pier E2 has more than sufficient capacity to resist the demands of the design SEE. This conclusion is further supported by the analyses in Sections 4.2.3 and 4.2.4 which indicate that the bearings will not be subjected to the 60 MN used in this evaluation under the design SEE.

4.4 Capacity of Strut

4.4.1 Introduction

The effects of the forces corresponding to load path C on the strut at Pier E2 were initially analyzed using the conventional method of strength of materials. A simple static frame analysis was performed to determine the internal forces of the strut. Critical strut sections with forces larger than those reported for load path B were checked according to the AASHTO LRFD Specifications. This assumes (per the scope for this review) that the components were adequately designed by the EOR for this load path. In addition, a lower bound load path wherein all the seismic force is assumed to be transferred through shear keys S3 and S4 was also evaluated.

4.4.2 Structural Analysis

The Lusas frame model shown in Figure 17 was subjected to the loading effects indicated by Table 8 for load path C. The eccentricity between the center of rotation of the bearings and shear keys and the strut centerline was considered in the definition of the model input loads. Table 9 presents the envelope of the sectional forces at the critical bearing and shear key locations including the gravity and axial post-tensioning effects of the strut. The red values indicated in Table 9 correspond to the demand forces that were more critical than their counterparts of load path B. Figure 18 through Figure 20 show the different force effect diagrams of the strut for load path C and the lower bound path.

4.4.3 Sectional Checks

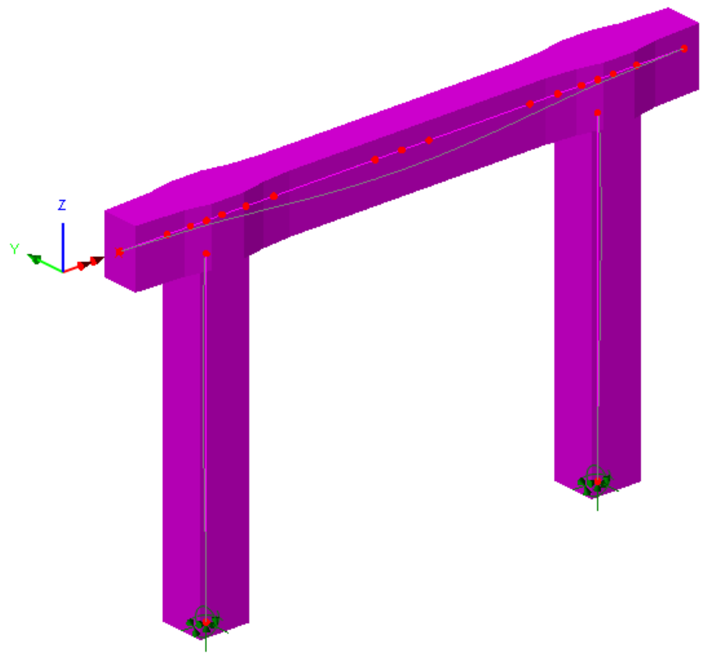
The critical forces from Table 8 and their concurrent effects were compared to the section capacities determined according to the AASHTO LRFD Specifications.

A summary of the principal check findings is presented in Table 9. The red numbers indicate the critical force effect taken from the envelope and the blue shaded numbers mean that the corresponding D/C checks are satisfactory. The complete calculations can be found in Appendix 4.

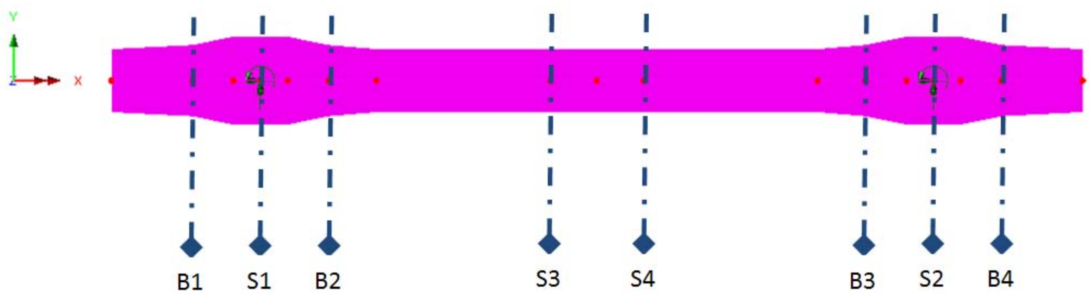
4.4.4 Results

The results showed that the maximum biaxial flexure interaction equation values were 0.91 and 0.94 at bearing B2 and shear key S2, respectively, while the demand-to-capacity ratios for shear were 0.82 at bearing B3, and 0.58 at shear key S2 (see Figure 17 for location of bearing and shear key sections) indicating more than adequate capacity to resist the SEE forces. The forces on the cantilever portions of the strut resulted in insignificant demands when compared to the available capacity of those components. In fact, the stresses in the cantilevered section of the strut based on this sectional analysis were found to be below the modulus of rupture when the post-tensioning effects were included, meaning the concrete would likely not even crack during an earthquake.

For the lower bound load path, although the minimum axial force in the strut was controlled by this loading case, the biaxial flexure interaction checks showed that this was not a critical condition. Between the columns, the demands are driven by the global frame-action moments and shears such that the change in forces caused by transferring the seismic loads through the bearings had a very minor effect on all but the axial loads, as can be seen in the comparison of the force effect diagrams between load path C and the lower bound path in Figure 18 through Figure 20.



a.



b.

Figure 17 - Overview of Pier E2 Frame model; a. 3D view. b. Identification of Strut Cross Sections

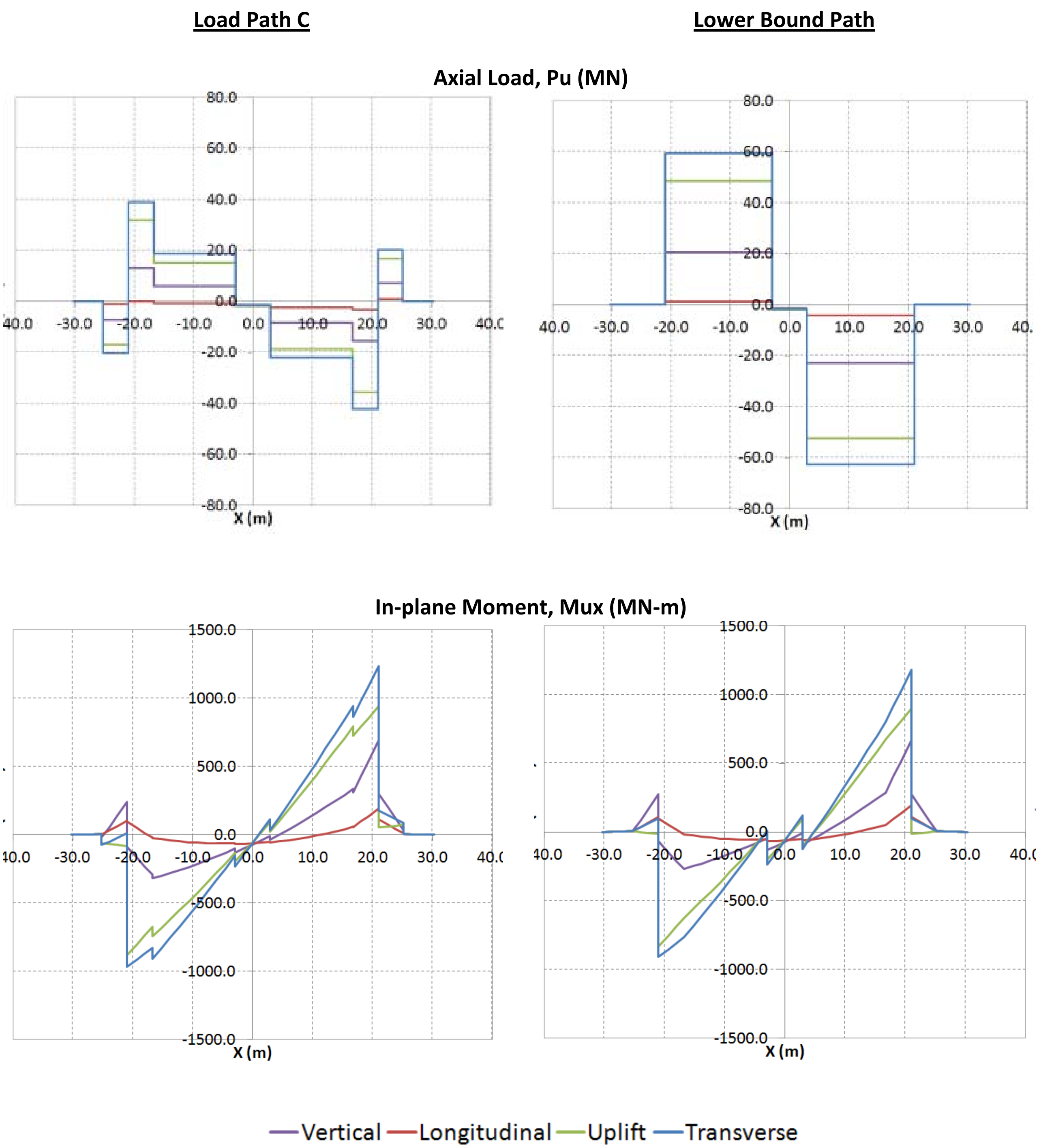


Figure 18 – Force Diagrams for Load Path C and Lower Bound Path (Note: axial post-tensioning effects are not included in the diagrams)

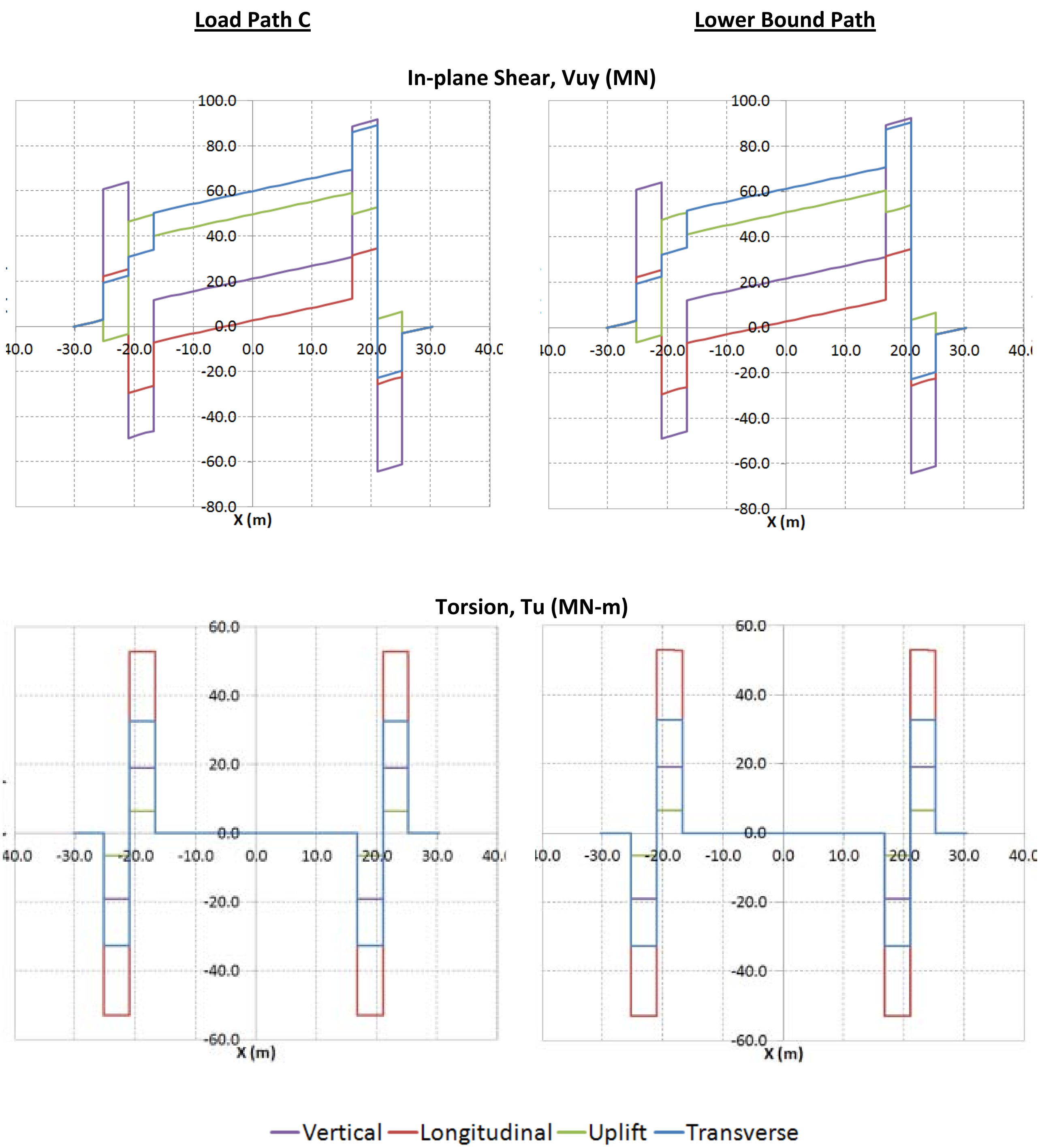


Figure 19 - Force Diagrams for Load Path C and Lower Bound Path

Load Path C **Lower Bound Path**

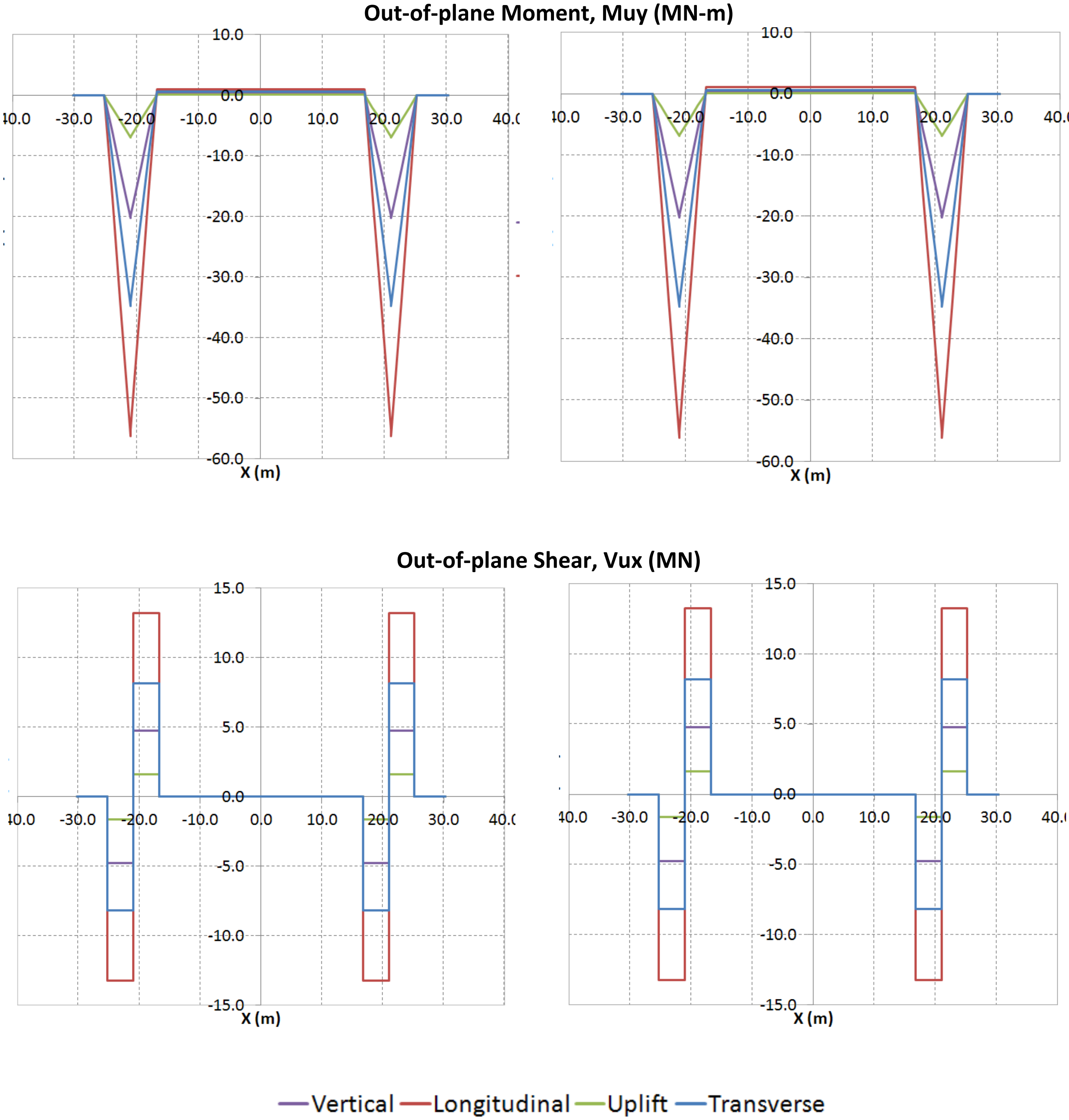


Figure 20 - Force Diagrams for Load Path C and Lower Bound Path

Table 8 - Superstructure loads on Strut at Pier E2

		Bearings B1, B2, B3, B4			Shear Keys S3, S4
		Transverse	Long.	Vertical	Transverse
		KN	KN	KN	KN
Load path B	Transverse	30496	8186	16441	0
	Long.	1340	13232	19255	0
	Vertical	10799	4770	57932	0
	Uplift	25287	1628	-9539	0
Load path C	Transverse	20331	8186	16441	20331
	Long.	893	13232	19255	893
	Vertical	7199	4770	57932	7199
	Uplift	16858	1628	-9539	16858
Lower bound path	Transverse	0	8186	16441	60992
	Long.	0	13232	19255	2680
	Vertical	0	4770	57932	21598
	Uplift	0	1628	-9539	50574

Table 9 - Envelope of sectional forces at bearings and shear keys

			Pu	Vuy	Vux	Tu	Muy	Mux
			Axial	In-plane Shear	Out-of-plane shear	Torsion	Out-of-plane moment	In-plane moment
			MN	MN	MN	MN-m	MN-m	MN-m
Load path B	Bearings B1, B2, B3, B4	max	-114	86	13	57	3	855
		min	-180	-61	-13	-53	-2	-797
	Shear Keys S1, S2	max	-118	89	13	57	-6	844
		min	-180	-52	-13	-49	-58	-1136
Load path C	Bearings B1, B2, B3, B4	max	-104	89	13	53	3	941
		min	-185	-61	-13	-53	-2	-912
	Shear Keys S1, S2	max	-104	92	13	53	-5	1232
		min	-185	-64	-13	-53	-58	-968
Lower bound path	Bearings B1, B2, B3, B4	max	-84	89	13	53	3	799
		min	-206	-61	-13	-53	-2	-770
	Shear Keys S1, S2	max	-84	92	13	53	-5	1177
		min	-206	-64	-13	-53	-58	-913

Table 10 - Critical Sectional Forces and Capacity Checks

Units: Force: MN Moments: MN-m Stress: MPa

			Bearings				Shear Keys		
		Element:	Bearings				Shear Keys		
		Controlling force:	Mux max	Mux min	Vuy max	Pu min	Mux max	Vuy max	Pu min
		Section:	B3	B2	B3	B3	S2	S2	S2
1.	Sectional loading effects:								
	Axial (if <0 → Tension)	Pu =	165	124	159	206	185	159	206
	(if >0 → Top fiber in tension)	Mux =	941	-912	304	799	1232	688	1177
	Out-of-plane bending moment	Muy =	3	3	2	3	32	18	32
	Torsion	Tu =	0	0	19	0	33	19	33
	In-plane shear	Vuy =	70	50	89	71	89	92	90
	Out-of-plane shear	Vux =	0	0	5	0	8	5	8
2.	Normal stresses assuming elastic behavior:								
	(if >0 → compressive stress)	fc 1 =	42	-29	17	38	44	26	43
		% f'c =	75%	53%	32%	68%	80%	47%	78%
		fc 2 =	-29	39	-6	-22	-33	-17	-30
		% f'c =	53%	70%	10%	40%	59%	30%	55%
3.	Axial and flexural force effects:								
		Pnx =	172	174	209	248	195	177	227
		Mnx =	1239	1274	1291	1341	1528	1497	1577
		ey ratio =	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		Pny =	1063	1209	1065	1082	997	845	1112
		Mny =	31	27	26	11	712	873	535
		ex ratio =	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		IE =	0.86	0.91	0.27	0.72	0.94	0.53	0.89
4.	Shear and torsional force effects:								
	Torsional effects:	ϕTn=	82	82	94	88	46	52	47
		Tu/ϕTn =	0.00	0.00	0.20	0.00	0.71	0.37	0.69
	In-plane shear:	ϕVn =	88	87	109	97	135	158	139
		Vu/ϕVn =	0.79	0.58	0.82	0.73	0.66	0.58	0.65
	Out-of-plane shear:	ϕVc =	51	51	51	51	63	63	63
		Vu/ϕVc =	0.00	0.00	0.08	0.00	0.12	0.07	0.12

4.5 Transfer of Horizontal Load From Bearing Base Plate to Strut to Column

Due to the size and localized loading conditions of the concrete pier strut, strut-and-tie models were developed for the two regions shown in Figure 21 as an additional check beyond the sectional interaction values of Section 4.4. Since the controlling loading case corresponds to the transverse forces, only two dimensional models subjected to these forces were considered in this study.

To determine an appropriate strut and tie model that accurately represents the flow of forces in these disturbed regions, an elastic analysis of a plane stress model of the strut was performed in Lusas to obtain the principal stresses in the uncracked regions. Strut and tie configurations were then developed based on the flow of forces in these models (see Figure 22). In order to bound the potential loads applied at the bearings and shear keys, the bearings were evaluated for the forces from load path B, while shear keys 3 and 4 were loaded with the lower bound forces, assuming all transverse seismic force is transmitted through these two shear keys. Results of the frame analysis were used to determine the boundary loads applied to the strut and tie models. Results of the analysis indicate that there is sufficient capacity to accommodate the changes in load path caused by the shimming of the bearings. One anomalous result was encountered regarding the tie capacity in the tension side of the pier column.

Since this is outside of the scope of this review, it was not pursued but was reported to the EOR for further review.

The complete calculations corresponding to the strut-and-tie models are presented in Appendix 5.

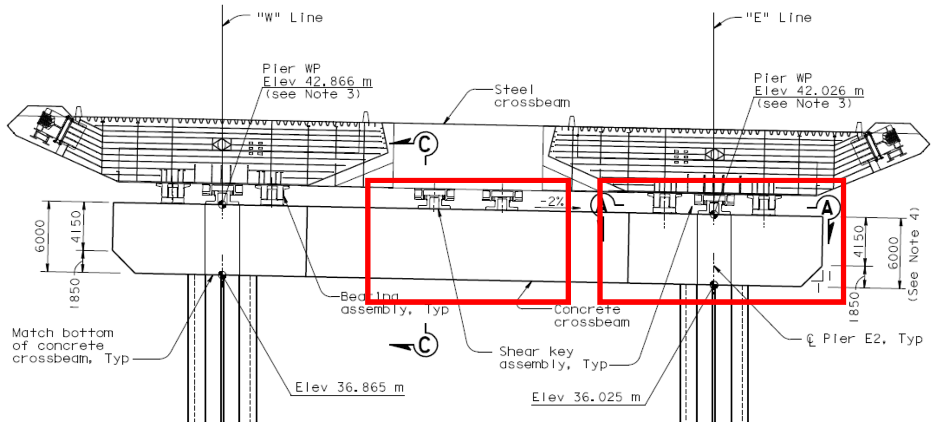


Figure 21 - Strut regions analyzed with strut-and-tie models

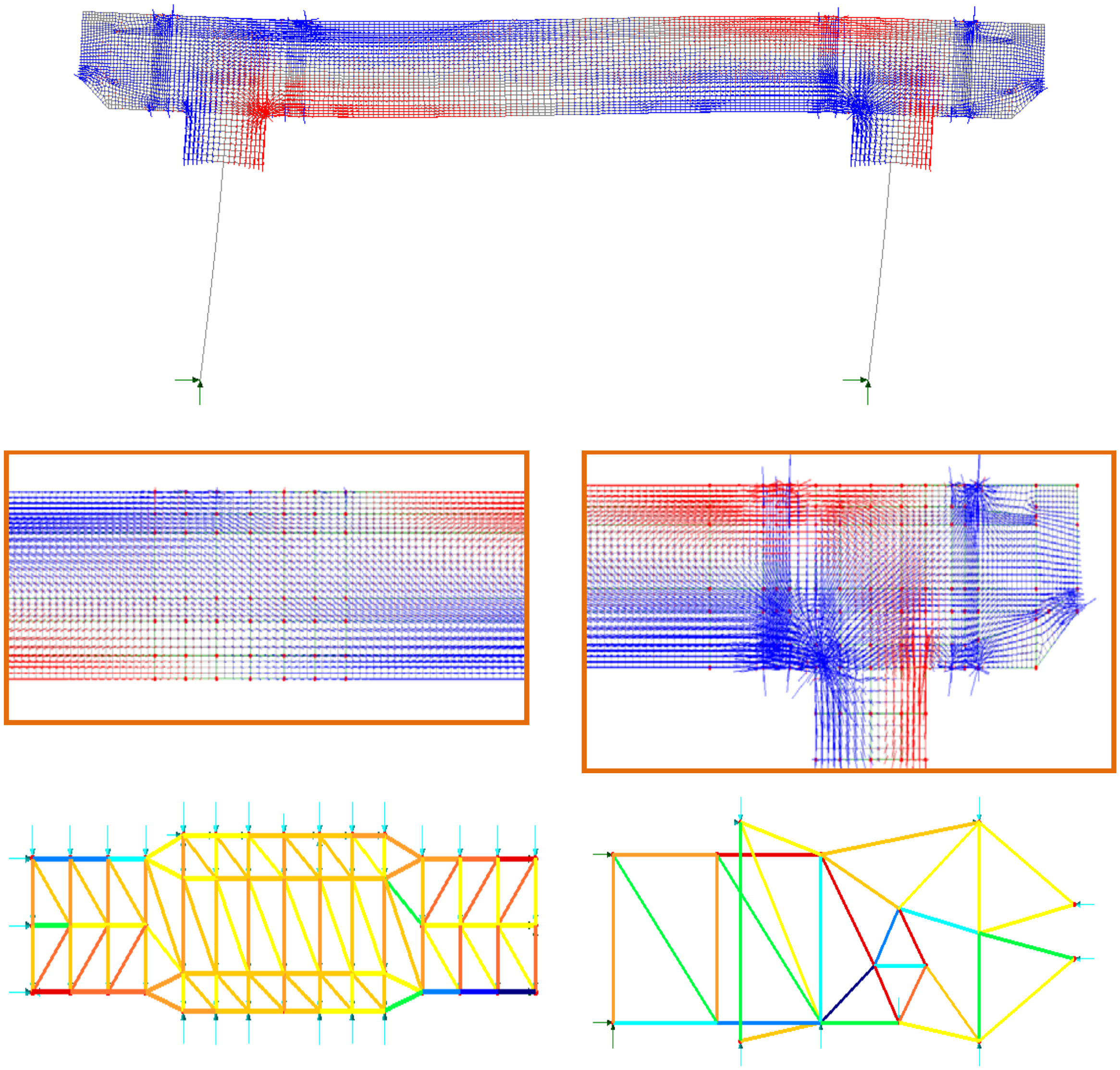


Figure 22 - Principal stress trajectories and strut-and-tie models (blue: compression, red: tension)

4.6 Potential to Damage the Permanent Bearings or Other Components if Shims are Installed

The analysis of Pier E2 in Section 4.4 shows that the service and seismic design forces in the pier are not changed significantly by the redirection of seismic forces resulting from the shimming. Therefore the response of the concrete strut and columns are not expected to be different with or without the shims. Based on this subjective assessment of the global effects, combined with the quantitative assessment of the local elements in the load transfer zone, it is concluded that there does not appear to be any reasonable possibility of damaging the bridge as a result of installing the shims, other than the possibility of damaging the paint system either when the shims are inserted or as the bridge rotates in service

5. Conclusions and Recommendations

The exact distribution of forces in the bearings and shear keys, and hence the demands on other parts of the load transfer zone at Pier E2 which is the scope of this investigation, is highly dependent on the planned and accidental tolerances (gaps) between various components. A precise quantification of the forces is, for practical purposes, unknowable. It is, however, possible to make estimates of the reasonable range of possibilities rather than placing reliance on a single solution; MM's assessment is based on that approach. Various assumptions have been used to place reliance on the set of shear keys and on the set of shimmed bearings, separately and in combination. Analysis of the resulting distribution of forces described herein formed the basis for the conclusion and recommendation below.

Modjeski and Masters, Inc. (MM) concludes that the concept of temporarily shimming the bearings at Pier E2 of the San Francisco-Oakland Bay Bridge as described in the July 15th information package entitled "Seismic Evaluation of SAS at E2 Bent Prior to Completion of Shear Keys S1 and S2", and the proposed details, will provide more than sufficient capacity between the superstructure and the strut at Pier E2, including the strut itself, to resist the design Safety Evaluation Earthquake. The service and seismic design forces in Pier E2 are not changed significantly by the redirection of seismic forces resulting from the shimming. Therefore, the response of the concrete strut and columns are not expected to be different. Other than the possibility of scratching the paint system either when the shims are inserted or as the bridge rotates in service, there does not appear to be any reasonable possibility of damaging the bridge as a result of installing the shims. Assuming that the rest of the structure has been properly designed, we conclude that the safety of the traveling public is improved by moving traffic on to the new bridge and we recommend that action.