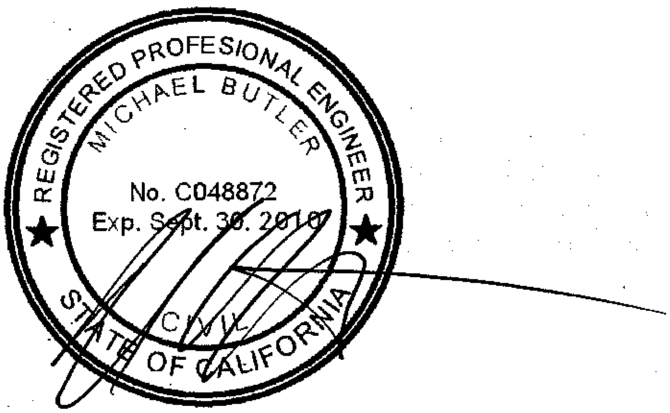
Shed #4

Structural Evaluation of Georgia Pacific Mill Site Storage Shed #4

August 17, 2009



Report Contents

Exhibits Attached

Shed #4 Plan and Section
Proposed Structural Retrofit Details
Retrofit Construction Specifications
Structural Calculations
Lumber Grader Notes

Please note that photographs are provided electronically.

Dorsey Design 249 North Main Street Suite F Fort Bragg CA 95437 964-6286

Summary

It appears that from a structural perspective, the integrity of Shed # 4 is such that the building is worth rehabilitating for reuse as an industrial art center.

If the "design" and "condition" aspects of structural integrity are separated, one could summarize that the design aspect is "generally as expected" and the condition aspect is "better than expected". The building has inherent value toward reuse that is primarily attributable to the original use of rot-resistant redwood for the major structural members, and the construction of a foundation that is above standard for the era and situation.

If the cost to retrofit Shed #4 is compared to that of constructing a new steel building, it must be kept in mind that the steel building must be built to the corrosion protection criteria necessary to protect exposed light-steel at a coastal site. Steel building manufacturers do tend to slight the true demands and cost of long-term coastal corrosion protection. As a reference perspective, a primer commonly specified for steel members on local coastal sites is described by the paint manufacturer as "for marine/offshore".

History

Shed #4 was built in two phases. The south half was built in 1965, and the north in 1967. The primary and most secondary structural members are of old-growth or good quality second-growth redwood. The minor structural members for the walls and roof are nominally 8' long 2x4's of douglas fir. This sub-framing of the building is all designed around the 4'x8' sheet of plywood, which presently dominates the external appearance of the building.

The primary roof members are site-built gambrel-roof-shaped trusses spanning 75', with bolted steel-strap connections onto 10x12 columns, at a spacing of 20' between trusses. These roof trusses each span half of the building width, creating a very long valley down the middle of the building, and so requires a very extensive cricket (carefully sloped area to drain the valley) running both north and south from the mid-point of the building.

The structural design criteria of Shed #4 is about as would be expected for a large storage building of the mid 1960's. More than forty years later, the building is continuing to serve its original purpose quite well. Assuming that non-structural architectural/code-compliance issues are all addressed, from a structural perspective Shed #4 can continue to serve the city's intended purpose for several decades more with the retrofit measures proposed in this report.

Our Investigation

Besides the engineer and building designers involved with this report, a lumber grader (Paul Johnson) and three contractors experienced in building renovation (John Koski, Bill Melville, and William Pell) were employed to make their observations. The reason for bringing these perspectives in is that so much of the building wood members' suitability depends upon their lumber grade well as their size, and of course they must be in sound condition. With limited access time, we made up for it with more eyes.

Because the purpose of this report is to provide information on the viability of the building for reuse as an industrial art center, the recommendations herein are in general realistic-to-pessimistic. All hardware and repairs indicated are in our opinion worst-case necessary retrofits that can be scaled back in many cases; however it is not out of the question that another engineer would be more pessimistic on any given issue.

This investigation went further in evaluating the structural condition of the building's members than our scope of work required, but it became apparent early that more of this information is essential to a realistic assessment of the viability of the building's rehabilitation. Fortunately some of the original design drawings were recovered by the City of Fort Bragg, and because we relied upon these to determine the building's foundation suitability it allowed us to spend more time evaluating the building's condition. It must be kept in mind that we still could only do random sampling of building member condition.

Structural Condition

Several experienced pairs of eyes pried into various corners of Shed #4 and they all drew the conclusion that it is in better condition than expected. Considering the age and initial appearance of the building, we all expected to find more rot and decay than we did with our random sampling. With the exception of the plywood siding, we found very little rot, although our equipment/access time allowed only limited checking high up on the walls and the ceiling.

Because much or the extensive cricket area does look suspect, and this type of roof drainage design tends to have problems in general, it would be prudent to assume that rot is present. Random sampling of primary and secondary roof members was undertaken, and this showed those members checked to be in generally fine condition – these members being primarily of rot-resistant redwood is of course a significant factor in their presently good condition.

Much of the area of the top of the exterior walls also looks water-stained and suspect. It appears that there was wind-blown leaking due to the lack of roof overhang, and due to roof gutters that are clogged with dirt and weeds, and therefore soaked water into the top of the wall. However, the little bit of checking that we were able to do in this area found no serious rot issue, but behind the gutters (where we did not have access) there may be a lot of rot. The use of regular plywood, with plain butt joints, as exterior siding may have also contributed to this leaking in the walls. In any case, it would be prudent to have this area checked by the contractor providing the retrofit estimate.

As the existing light framing is all 2x4 douglas fir, it is important that those roof members (sub-purlins) are checked for soundness, as a safety issue to workers on the roof.

Rusting of steel connections is obviously present, but it appears that few of the connectors or their bolts require replacing for that reason. We did not remove any of the hardware to see if there is significant corrosion on the backside against the redwood, or of the bolts inside the redwood. We are making an assumption that the hidden corrosion it is typically not a threat to necessary structural integrity, and that the new use of the building will provide a semi-conditioned environment that will slow further such corrosion.

Removing and replacing the connectors that are at each end of the trusses would be tedious and expensive because of their integral design. The retrofit of these connections is designed to add to their strength with new hardware (without trying to weld to the existing steel), and this retrofitting can be done more extensively where the existing steel is more deteriorated. One example of such a connection that is significantly deteriorated is right at the roof access ladder, where a large bird nest has been present, apparently for years, right above the connection between trusses and a column. This is an example of a special case that is also at a wall line, so it requires less strength (if the shear walls are balanced on both sides, etc). If it were a connection that requires more strength, more retrofit strength could be added, for example.

For the typical situations, all steel connectors will require in-location sand-blasting and rust-preventative painting, per the attached specifications, unless another viable option is presented. All of the plywood siding that will remain will require renailing because of rust to those nails (as well as varied deterioration of the plywood).

The plywood siding has taken a beating, but most of it appears that it will be ok if renailed and treated with borate solution or the equivalent. It is assumed here that new felt paper and siding will go over all of the existing plywood, if it is confirmed that drying out and borate treatment will stop further fungus growth. Our opinion is that this will typically be ok. To be safe, also budget a subsequent treatment with "copper green", when the existing plywood edges are again dry enough to absorb it. Fortunately most of the grade around the building is paved, allowing the use of a wheeled scissors-lift for this work.

For costing purposes, one should assume that both end walls will have all plywood replaced – although this is probably pessimistic, especially for the north wall, and it is probably moot anyway, given the significant architectural changes that are likely at those locations. This proposed architecture will dictate structural retrofit cost, and depending on that design, this may be a situation where seismic loads control the retrofit cost.

With regard to the long walls, assume that the east wall will require all of the lower course of plywood replaced, and that the west wall will require around a third of the lower course replaced; those of the lower course that are not replaced will need the excessive overlap with the concrete stem wall to be trimmed back to the bottom of the mudsill, and the bottom edge treated with "copper green" (I do this on all construction). Assume that for both of the east and west walls, the portions of plywood between each of the clerestory lights will need to be replaced. Assume a new vertical 4x8 sheet between each light, with all existing plywood above and below the lights to remain. The configuration of the lights is likely to change anyway.

With regard to the crickets, the south one seems to have had more leak problems than the north. For the south, one should pessimistically assume that all the framing and plywood in the entire cricket area needs replacement, in order to come up with a safe cost estimate. Even if this area proves to be rot free, it has a history of leaking and so the reasons for this must be determined and corrected. If a roofer is not willing to guarantee the existing cricket design (of minimal slope) to be leak free, then new crickets with more slope would then be required for both the south and north side.

The north cricket appears to have had few leak problems. It is not clear as to why it has had fewer problems than the south, as it has no more slope than the south cricket. Any rot found in this area would have to be repaired, regardless.

Floor, Foundation, and Grades

As it turns out, the floor in Shed #4 is not level, but it largely follows natural grades. The northeast corner is four feet higher than the southwest, which is a big difference, but it is less than 1% slope (0.86%) over that distance. While this may help with drainage if the roof should leak, it is of course not what the Industrial Art Center tenants would expect. The installation of a new level slab, over the existing paving, that steps down according to partition locations or other architectural features, is what is anticipated. This new slab is designed as an essential element of the structural retrofit in adding required weight to the footings so that the building cannot blow away.

However, pouring new slabs throughout the building would not have to be necessary from a structural perspective if there are uses of the building that would not require a new level slab on grade. The existing asphalt paving is in good enough condition for forklifts, etc, as can be seen from the photos. In lieu of a new slab, the required anchorage could be accomplished by drilled piers adjacent to and connected to the existing column piers. Or, if a mezzanine structure is planned for part of the building, this steel/concrete structure can be designed to provide necessary wind anchorage, as well as bracing for the existing building.

Outside the building along the north portion of the east side it will be necessary to remove soil to get better clearance between wood and soil and to allow better drainage around that portion of the building. This soil can be used for fill inside the proposed new slab where appropriate to make the necessary level steps, although the contractor may prefer to import 100% sand because the compacting is more straightforward.

From the available design drawings for Shed #4, the foundation appears to be built with a relatively good amount of reinforcing steel, considering the era. The steel is less than would be used today, but this would be made up for with the installation of the new epoxied-in anchors, set quite deep. The quality of the concrete appears to be very good – as good as the average ready-mix delivered in Fort Bragg today. We did not notice obvious cracking, spalling, or signs of deficient concrete or reinforcement corrosion.

The existing concrete does show significant efflorescence (mineral deposits) on the interior surfaces, and this could be a sign that salty moisture, which settles on the adjacent paving, is being absorbed. As the efflorescence does not appear out of the ordinary for footings exposed to moisture, and the design drawings call for 3" of concrete cover over the reinforcing (at least for the exterior faces – it is unclear if this also applies to the interior). Accordingly, there does not appear to be reason for concern about corrosion of existing reinforcing, but we are not experts in this field.

Structural Design of Shed #4

Being an expansive, exposed, and relatively light building, Shed #4 is relatively more affected by wind than earthquakes. It was expected that the original wind design for Shed #4 would be according to (effectively) lower design loads than are currently accepted, and this has proved true, primarily in terms of structural member connections.

One disappointment was the sizing of secondary members; the purlins (roof beams between trusses) are marginally appropriate for their 20' roof span, and also girts (wall beams that span between columns) are undersized for their 20' span of wind loads. The primary framing members: main truss members and their support columns are more within the parameters of contemporary design – with the exception of their connections, which are repairable. Additionally there is no significant problem with the design of the minor framing members (short span 2x4s). However it must be kept in mind that the existing roof is not designed for new additional weight, as could be an aspect of fire separation requirements of some possible mixed uses of the building.

Some good news is that seismic retrofit is not a significant cost issue. With the existing architectural design, it can be assumed to add almost no cost if the wind design is properly addressed. Changes to architectural design that add weight would also add to seismic retrofit requirements.

This evaluation of Shed #4 is primarily of the construction of the building that is repeated regularly, and these elements are evaluated carefully. Atypical situations are not addressed, and portions of the building that will certainly be modified or affected for future architectural reasons are slighted. The cost estimates for these areas can be extrapolated from those of the typical portions that are carefully addressed. Specifically, the end walls have structural design issues that will primarily depend upon the final architectural design for those walls and including the existence and locations of interior partitions, and so the existing structural design of the end walls is not totally relevant. Similarly, 2 steel beams have been retrofitted with new steel columns, to allow the removal of 2 existing interior columns. These were not investigated, but appear to be ok.

Structural evaluation of the building was done by using the currently-enforced 2007 California Building Code as reference criteria (the 2007 CBC is based upon the 2006 International Building Code), with the assumption that the proposed Industrial Art Center is to be a building of normal importance according to classifications of the CBC.

The presently-enforced codes do not mean that historical performance is totally moot. Shed #4 has performed satisfactorily since 1967, and this proves that the original design has worked for 40 years at least. While performance in a design event earthquake or windstorm that did not occur over those 40 years is not an aspect of structural design that can necessarily be extrapolated, the performance of roof members that have shown to resist gravity over that time period can be expected to continue to do so in the future, even where they do not meet current code-required gravity loads.

This point is being made because many of the roof members - the purlins spanning between the trusses, depending upon lumber grade, for example - do not technically meet current roof design loads, yet they do not necessarily show excessive distress in over 40 years. Similarly the wall girts spanning the wind loads between the columns do not meet wind design load requirements, if one assumes that the building is “semi-enclosed’ as is (arguably) the current configuration; however their failure will not cause the building to collapse, providing that the primary members are designed properly. The question is whether the marginal members need to be supplemented or whether they can safely be left as is, based upon 40 years of performance. For costing purposes, it is assumed that those members that are of relatively lower lumber grades, or are damaged, or show signs of distress, would be those needing to be doubled up with additional new members. Where the original members are in good condition and are of adequate lumber grade, then they would be left as is. We do have concern about the varied appearance of some original and some doubled-up members, but we don’t have the cost-effective answer for that.

The notations from the lumber grader are attached as an exhibit to this report.

The existing foundation design appears to have used soil loading assumptions that are not significantly different from contemporary ones. Most likely, in a new building of this size, there would be a grade beam connecting the interior columns, and the perimeter footings / grade beams would in general be heavier. However there does not appear to be obvious foundation distress, and our retrofit approach does address these design issues, as there does not appear to be problems with the foundation design other than a lack of adequate dead load for currently-enforced wind-anchorage design loads.

Discussion of the Proposed Structural Retrofit Details

These details are proposed as the most expedient means of satisfying the code design loads, but they may have architectural/aesthetic compatibility issues. For this reason, some other options (likely more expensive) are also offered. The wind loads are applied generally pessimistically, and the typical deficiency found is in the connections between primary framing members: roof trusses and their supporting columns, and those columns and the foundation. In many cases smaller hardware elements than those specified could be used, but for practical costing purposes the hardware specified will do.

As mentioned above, it is anticipated that special architectural attention will be given to the building end walls, and at the few locations where the typical building structural pattern is interrupted with atypical conditions, and the new architectural design will trump any structural retrofit of the existing architecture of these locations. For this reason, little attention is paid to these areas of exception to the general structural pattern. For costing purposes, if no architectural changes are made to these areas, one could double the cost per lineal foot of the typical retrofit, to come up with a safe approximate cost of the retrofit of these areas.

Example specifications for this retrofit construction are also attached to this report.

If the structural retrofit of Shed #4 should go forward, of course more thought can go into these structural retrofit details, and of course more new details would be necessary to address the various specific situations. These details should be sufficient to determine the viability of retrofit and reuse of Shed #4. They are all drawn at a scale of 1½" = 1'.

Detail 1: Exterior Foundation and Wall Retrofit

According to the existing building construction drawings, the existing foundation footing size varies in that the north half of the building has footings only 8" wide (apparently), rather than the normally accepted 12" wide. Even if this unusual condition actually exists, the retrofit recommended will suffice – these are generally non-bearing walls.

The new slab is to be poured, and it is to be epoxy-doweled into the existing foundation wall. This practice – bonding a new slab to an existing foundation - does promote some cracking to the new slab, which can be reduced to acceptable amounts by proper employment of full-thickness control joints. To reduce new slab cracking to limits within "architectural slabs" more expensive retrofit methods are possible, but have not been designed by us.

The existing anchor bolts are essentially doubled, and the existing wall girts are doubled up where necessary, per previous explanations.

Detail 2: Exterior Foundation Pier and Column Retrofit

The primary requirement at the base of the existing 10 x12 redwood columns is to anchor them for wind uplift. The least expensive hold down that satisfies the worst-case design load is the HDU14 specified, a $45 piece of hardware not including the anchor rod. The HHDQ14 is more attractive and appropriate-looking for a building such as Shed #4, and is around twice the price. It must be kept in mind that that all of the exposed hardware for this building is specified to be hot-dip galvanized or alternative corrosion protection, which is not included in these prices. More explanation is on detail 3 below.

Detail 3: Interior Pier and Footing Retrofit

The explanation for detail 2 applies here, except that a custom-made hold down option is also shown. This is to show a lower-profile custom-made hold down that should not be expensive, and may suit the existing industrial architecture much better. This approach would be appropriate for any of the retrofitted connection locations. Heavier lag screws could also be substituted in order to keep in line with the existing appearance, where the new hold downs would be exposed.

At interior pier locations the new slab is heavily reinforced to give it the strength to pick enough of itself up to provide the required dead load to prevent the interior columns from uplifting (also to help confine the existing pier concrete). Where grades allow, the slab can alternatively be thickened to achieve the same result. Where a new interior footing will be placed in line with an existing pier, it will serve this purpose. Special considerations for control joints are required – in general they need to be around 10' away from the interior columns for this approach to work.

Detail 4: New Interior Wall and Footing (Aligned with Existing Truss)

Assuming that the steps in the new slab and new interior partition walls can align with existing trusses, this would be the detail for this condition. If the new interior wall is perpendicular to the trusses, then it can connect from truss to truss. In any case, the bottom of the truss would have to be braced to laterally support the top of the new wall – on example of this is shown on the detail.

The footing trench is cut through the existing paving. The footing size shown is probably pessimistic, and may be smaller in practice, depending upon the shear wall requirements of this detail. Use this detail for cost estimating in any case.

Detail 5: Truss Exterior Heel Retrofit

This is the most vulnerable connection of the building, as in the typical case it is insufficient in all 3 dimensions, and it is the most vulnerable to wind damage even if designed to code-loads. In the vertical direction, the column hold-down must be placed to take care of net uplift, with the anchorage being to a steel bar-washer slid into a notch carefully cut into the top of the truss top-chord member. This is required at all trusses.

In the plane of the wall, the top member of the wall ("top plates") that breaks at the truss must be tied to the corresponding member at the other side of the truss. This connection requirement will vary (reduce) according to the interior partition locations. The worst case condition is assumed in this detail.

For loads transverse to the wall, the truss bottom chord connection needs some help for the span requirements. This retrofit may be necessary even at interior partitions, so consider it typical.

Detail 6: Truss Interior Heel Retrofit – Typical of Both Sides

The truss on each side of the interior column needs the hold down for wind uplift. The delicate part of this retrofit is that the vertical anchor rod must penetrate the existing ¾" plate with enough edge distance to give it horizontal anchorage to the steel strap that connects to the truss bottom chord, both sides of the column.

Detail 7: Truss Middle – Retrofit

At this location the truss bolted connection is overstressed and the truss bottom chord member itself is slightly overstressed, if the material removed by the existing bolt holes is considered. Thus a wood tie is designed to connect in both directions outside of the existing steel connection in a manner that will reinforce the existing connection and the truss chord tension strength. This detail could be ignored at interior partition locations.

Detail 8: Retrofit of Faulty Purlins

This detail is based upon the idea that the 40-year performance of the existing purlins should be a legitimate factor in determining how many of them to reinforce with new parallel members. For contemporary design of a building in Fort Bragg with the geometry of Shed #4, the typical combination of purlin size and lumber grade present on Shed #4 would not be specified – they would be one size bigger. However, most of the purlins appear to have performed satisfactorily over their life to date, with a fire sprinkler system present. Unless the proposed new use of the building will place additional burden on these members, most of them should be ok as is, in the quantity noted on the detail.

It is assumed that all of the purlins will need the additional connection specified for wind uplift loads.

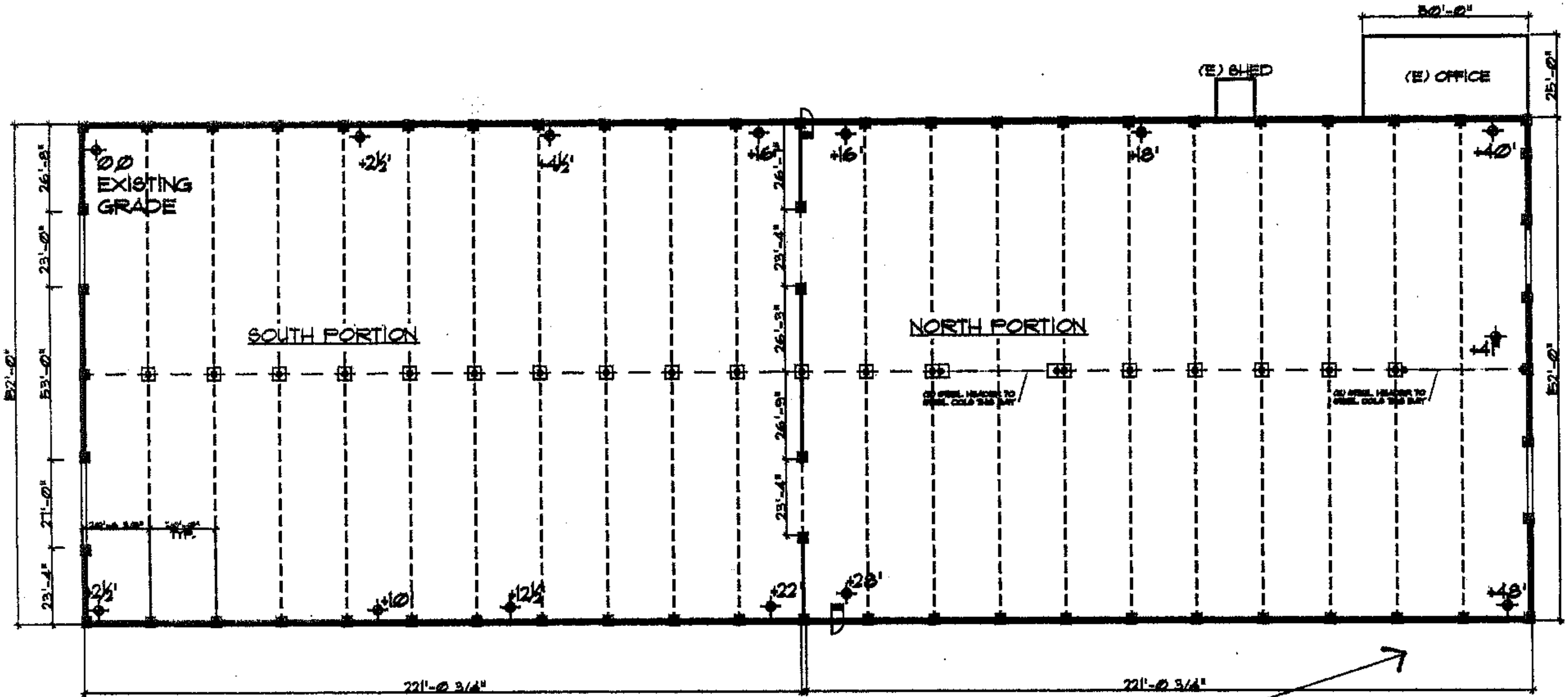
end of report body text

GP MILL SITE

STRUCTURAL REFERENCE PLAN:DRY SHED #4

KEY

- 16 — BAY GRID
- △ — (E) GIRTS, REPAIRED
- 15 — TRUSS GRID



CUT (E) GRADE & SLOPE AS
REQ'D FOR PROPER DRAINAGE

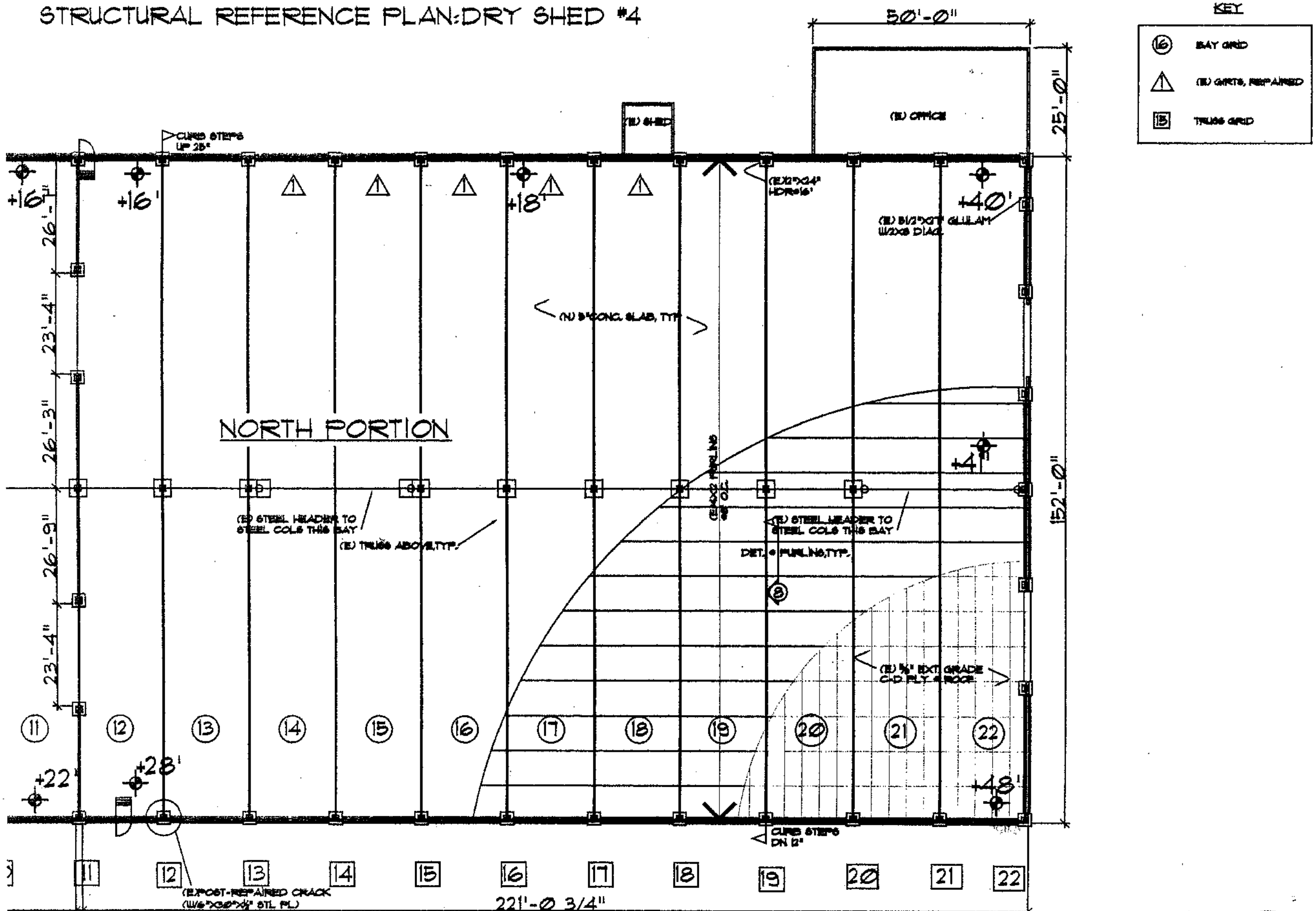
N

PLAN

1"=50'-0"

GP MILL SITE

STRUCTURAL REFERENCE PLAN: DRY SHED #4

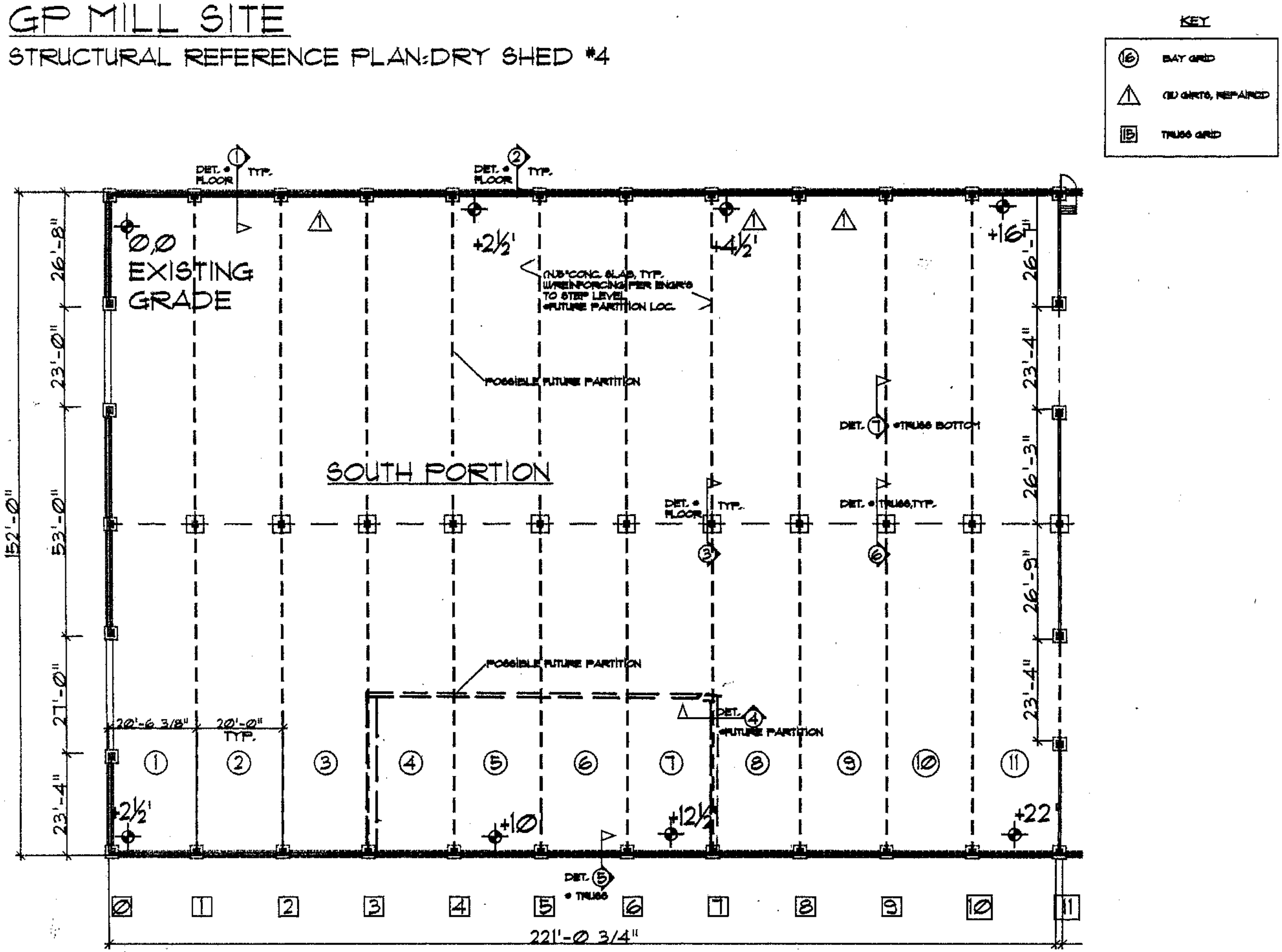


NORTH FLOOR PLAN

1"=30'

GP MILL SITE

STRUCTURAL REFERENCE PLAN: DRY SHED #4



SOUTH FLOOR PLAN

1" = 3Ø'

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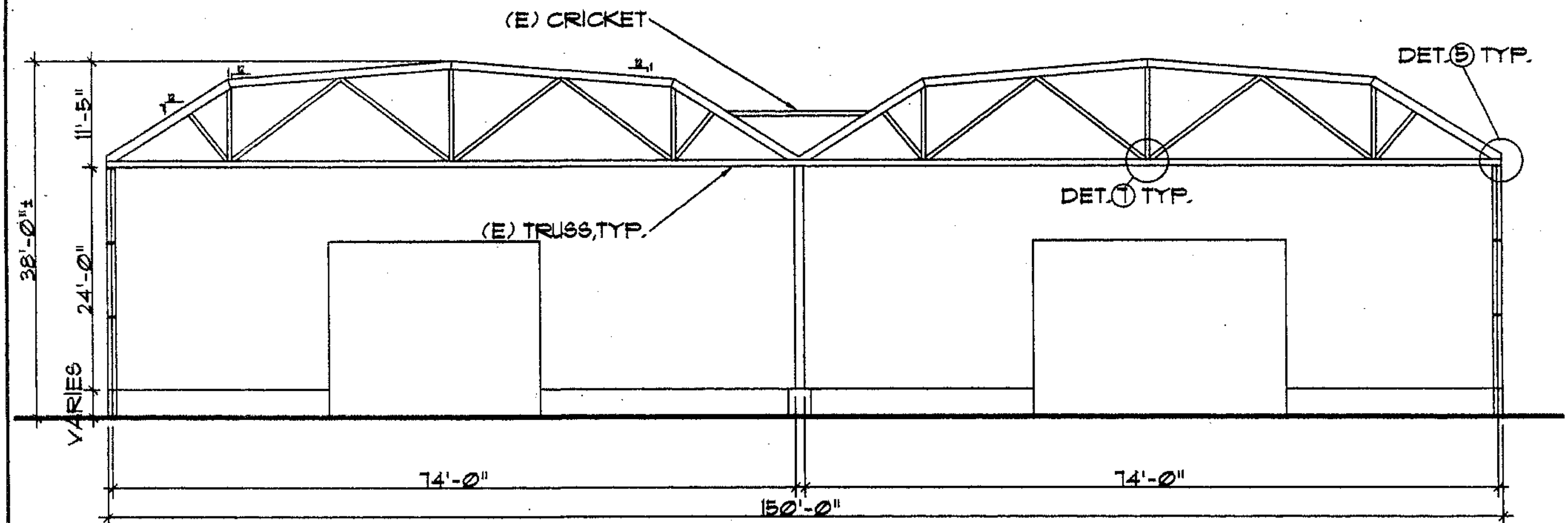
STRUCTURAL REFERENCE PLAN: DRY SHED #4



SECTION SHED #4

1/16" = 1'-0"

Michael Butler Civil Engineer
PO Box 1520 Fort Bragg, CA 95437 (707) 961-1891

Shed #4

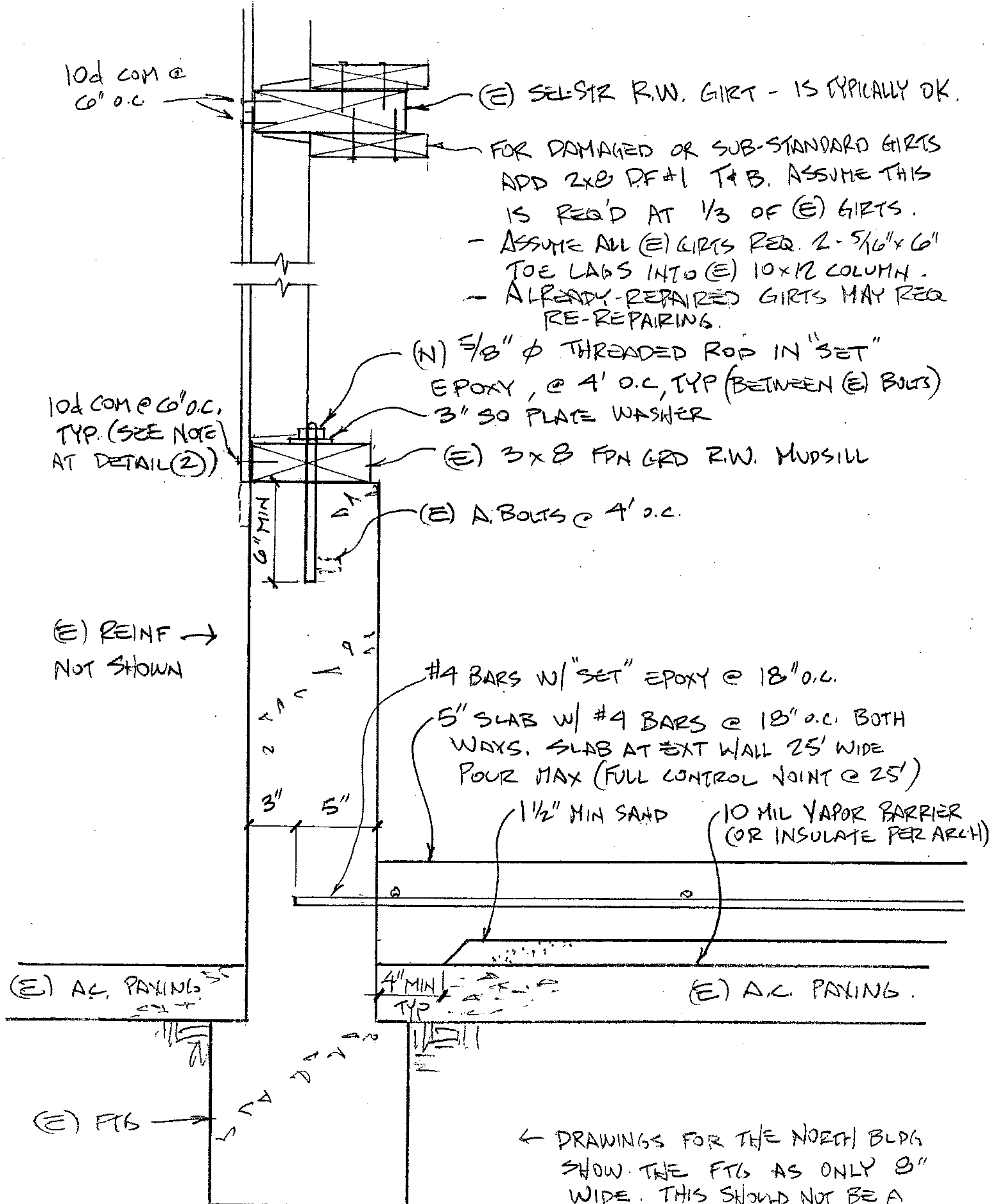
Structural Retrofit Details for Storage Shed #4

Details 1 through 8 are attached. All are drawn at 1½" = 1'

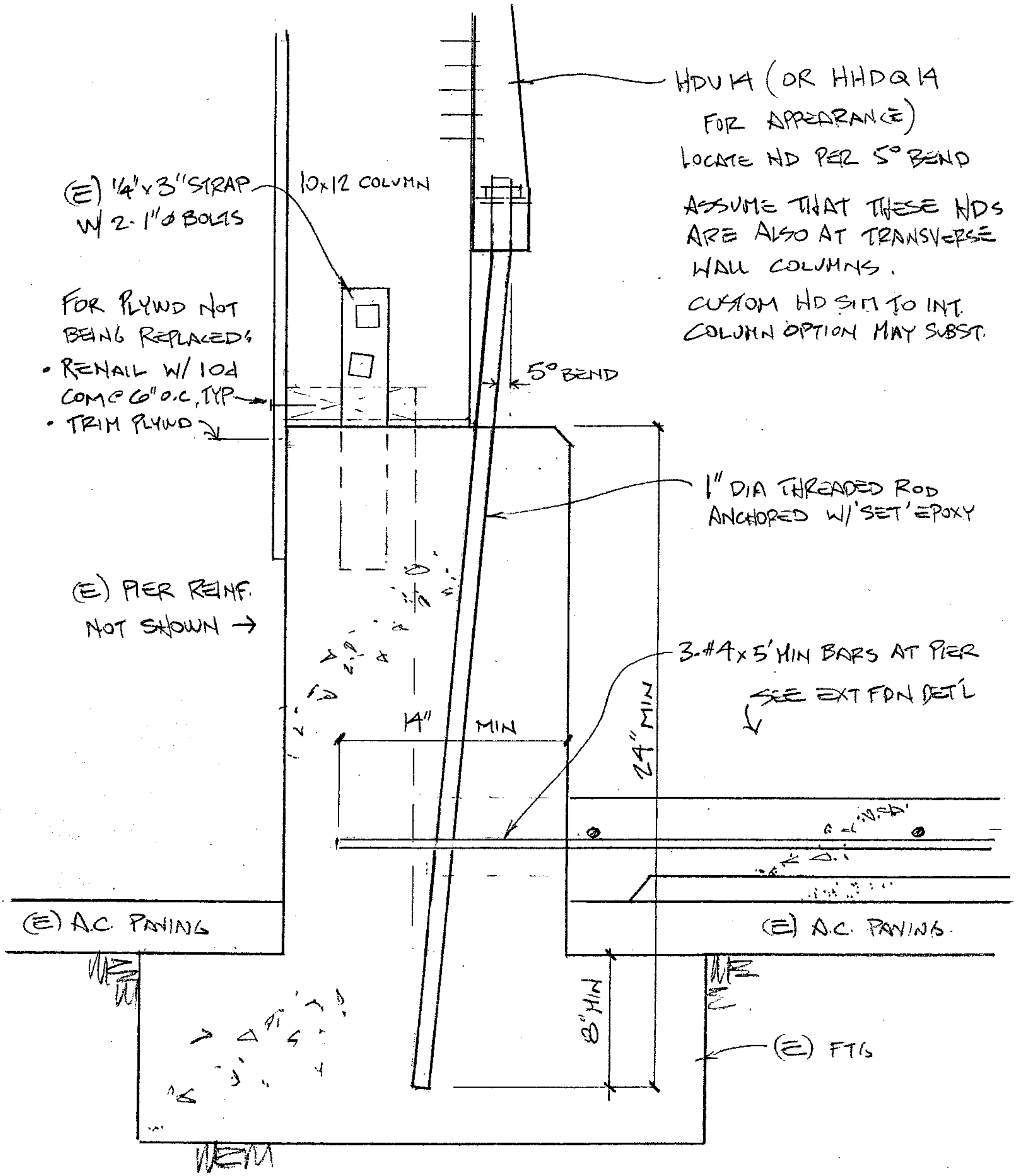
Notes for these details are in the Structural Evaluation Report

August 10, 2009

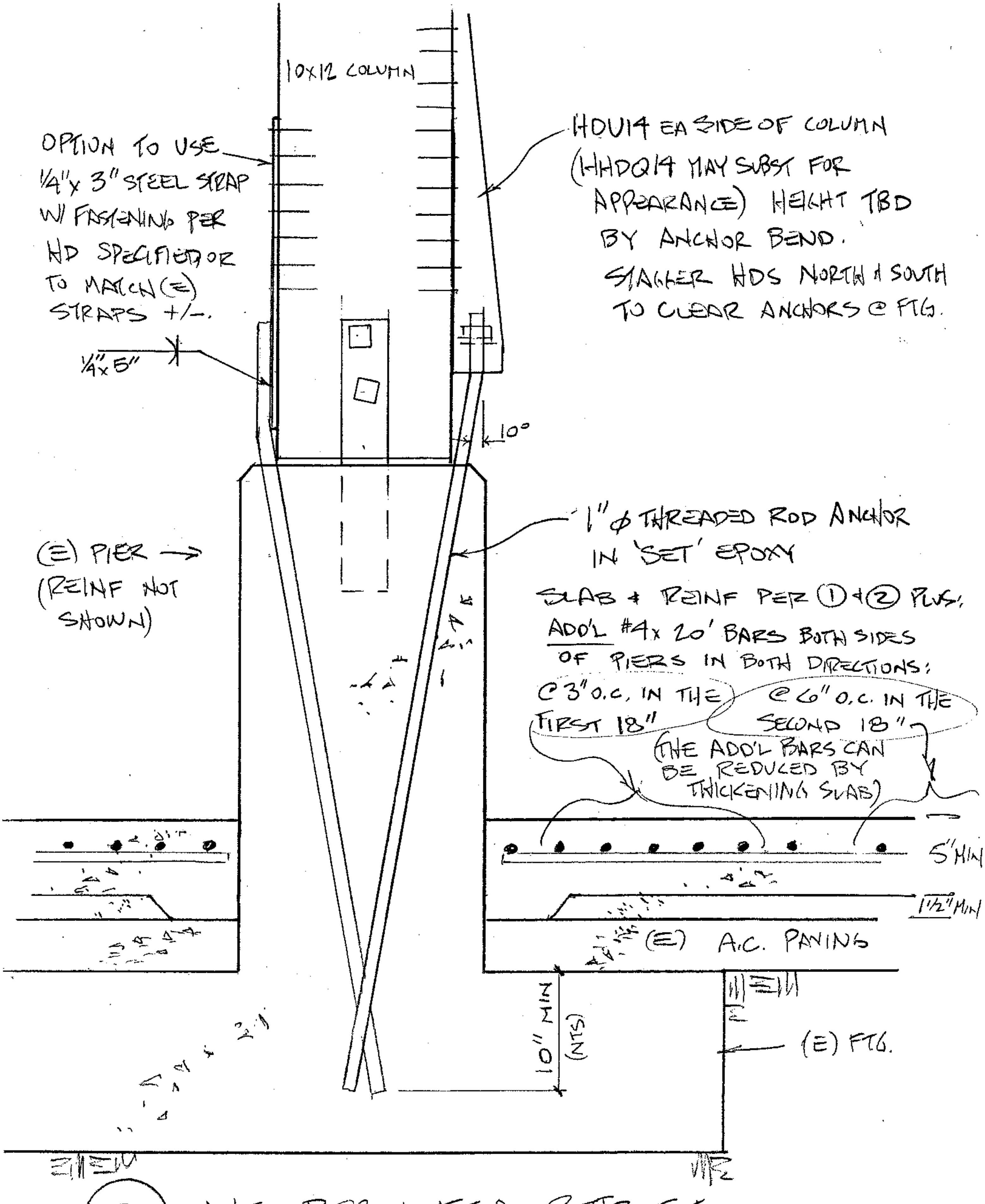
REGISTERED PROFESIONAL ENGINEER
MICHAEL BUTLER
No. C048872
Exp. Sept. 30, 2010
CIVIL
STATE OF CALIFORNIA



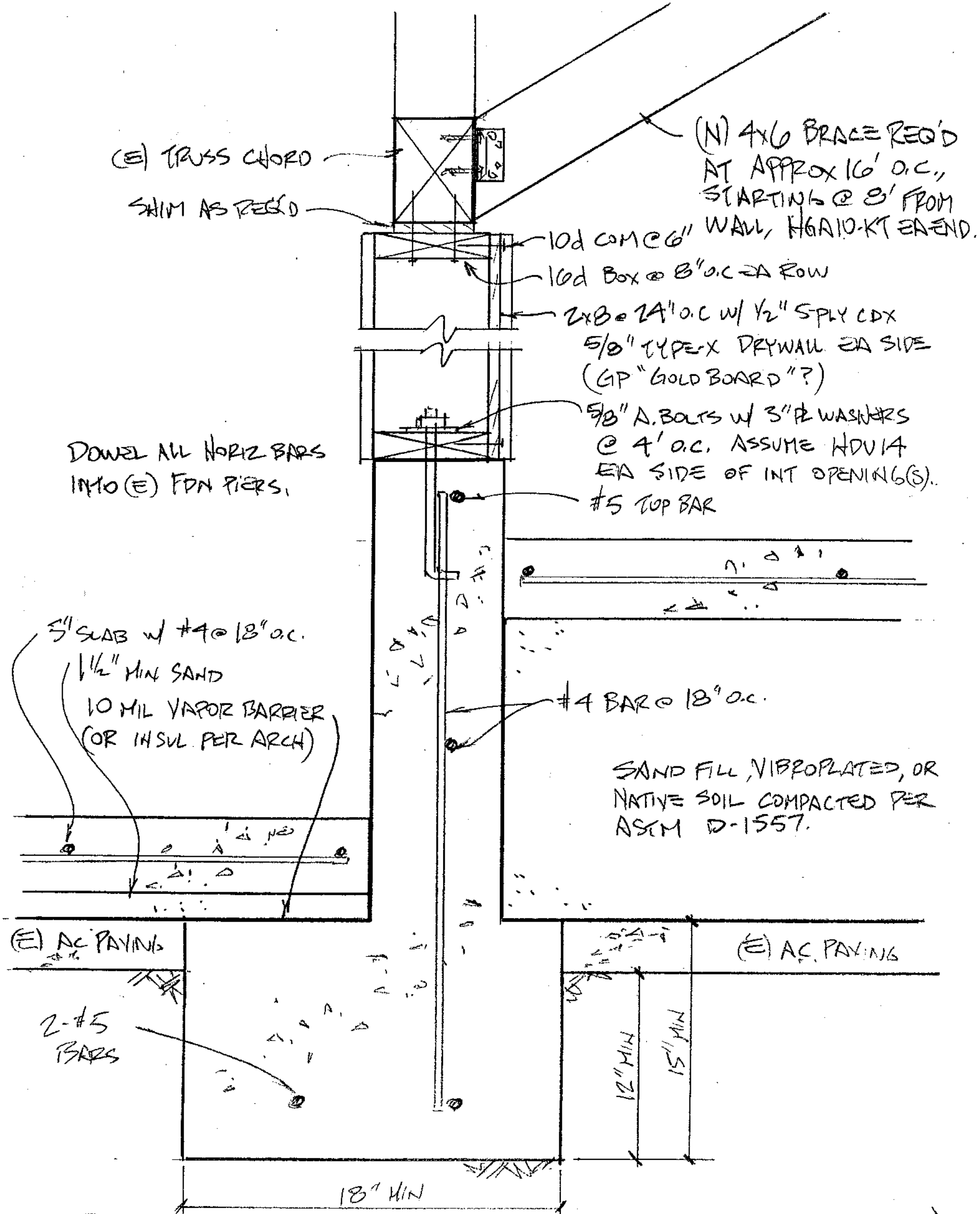
1 EXT FDN & WALL RETROFIT



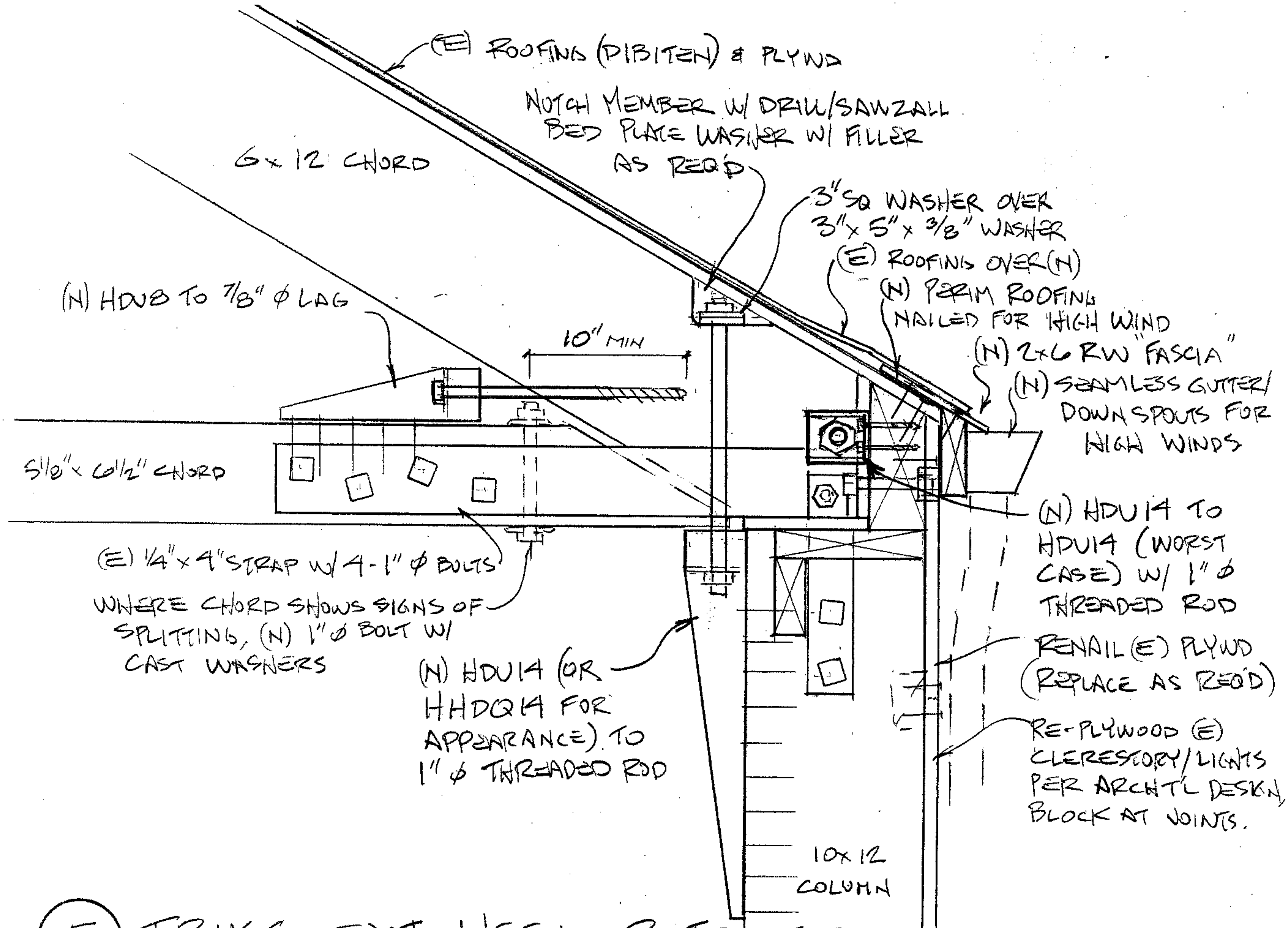
2 EXT FDN PIER + COLUMN RETROFIT



3 INT PIER + FTG RETROFIT

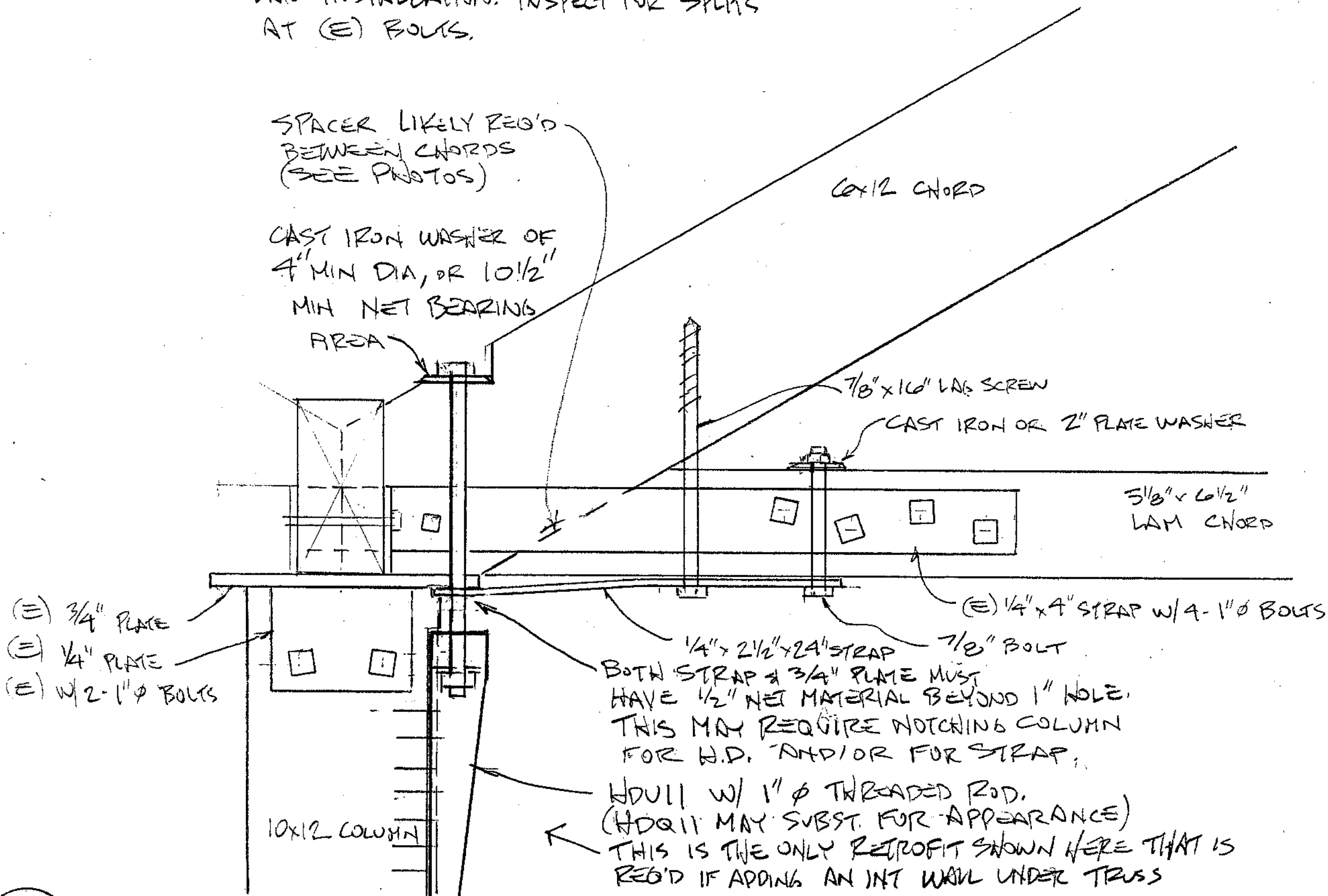


4 NEW INTERIOR WALL + FTG (ALIGNED W/ (E) TRUSS)

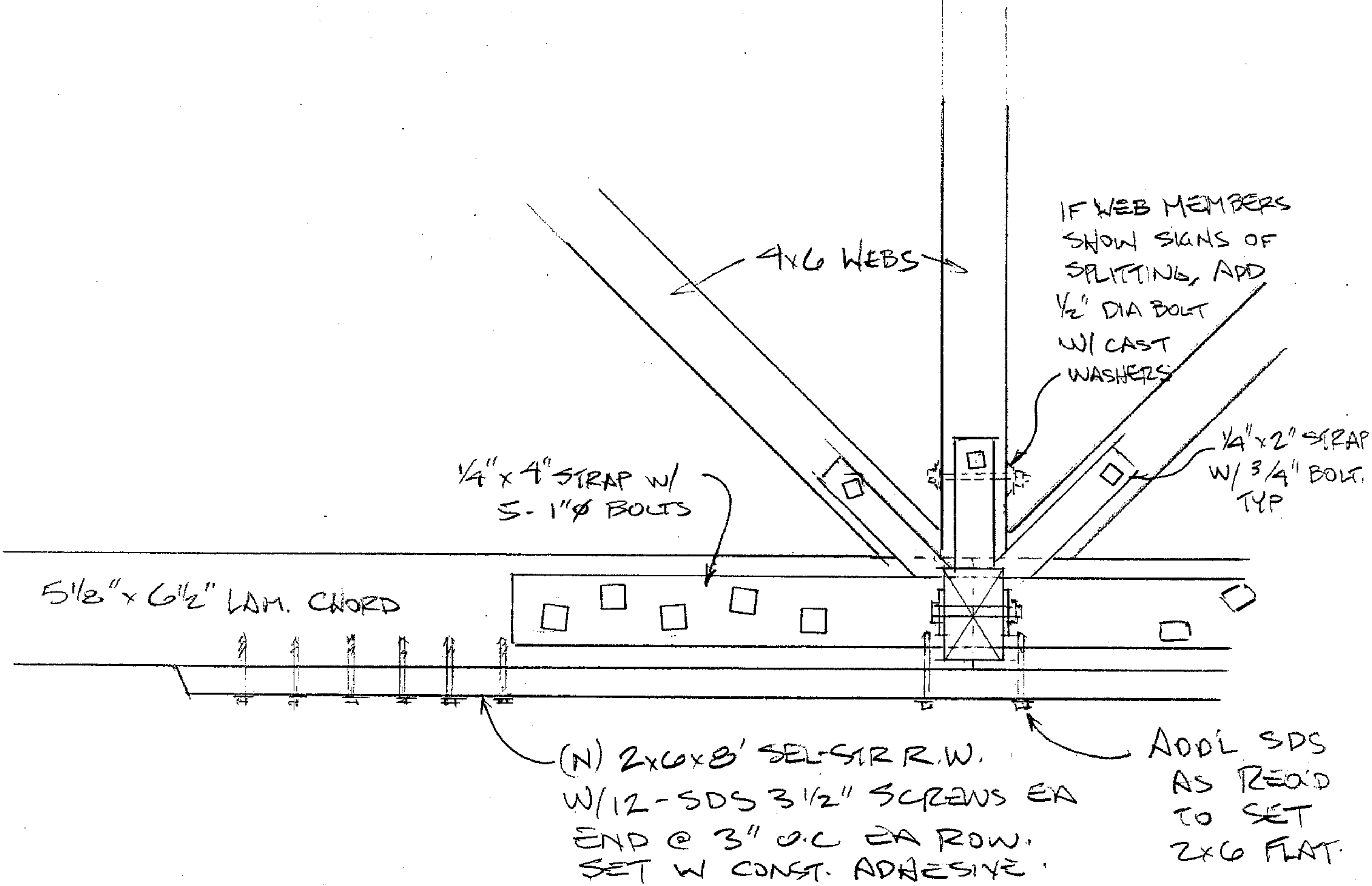


5 TRUSS EXT. HEEL RETROFIT

NOTE: BTM CHORD MAY REQ. LIFTING PRIOR TO LAG INSTALLATION. INSPECT FOR SPLITS AT (E) BOLTS.

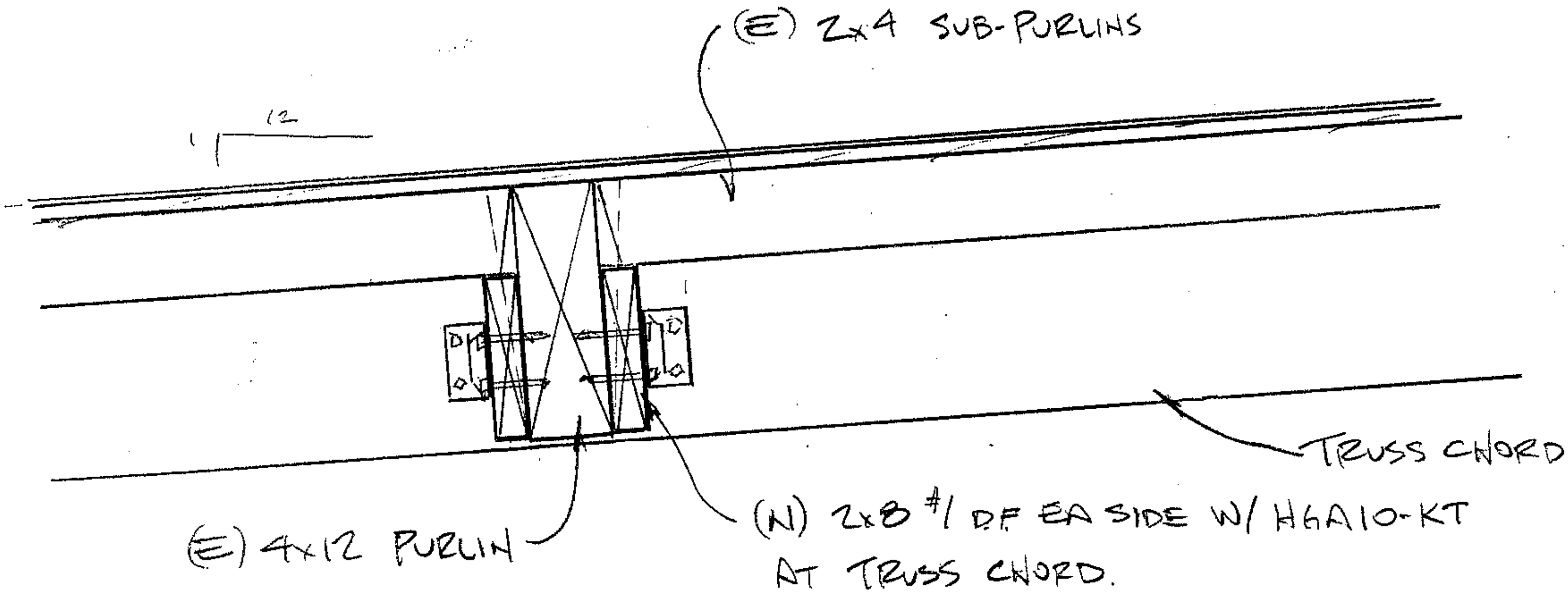


6 TRUSS INTERIOR HEEL RETROFIT - TYP OF BOTH SIDES



7 TRUSS MIDDLE - RETROFIT

THIS RETROFIT IS NOT REQ'D WHERE ADDING A WALL UNDER THE TRUSS.



THIS IS REQ'D ONLY AT APPROX 15% OF (E) PURLINS.
ASSUME REMAINDER OF PURLINS REQ. 2-5/16"x6" LAG
SCREWS INTO TRUSS CHORD (CAN SUBST. 1/4"x6" SDS),
FOR WIND UPLIFT ANCHORAGE. (PURLINS @ TRUSS JOINTS OK)

8 RETROFIT OF FAULTY PURLINS