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ESK-96-041

ComEd

April 1, 1996

U.S. Nuclea· Regula ory Commission
Attn: Documer. Cr.ntrol Desk
Washington, D.C. 20555

Subject: Quad Cities Nuclear Power Station, Units 1 & 2
Commonwealth Edison (ComEd) Response to NRC Request for Additional Information on March 28, 1996, Regarding Unit 2 Corner Room Steel Operability Evaluation

NRC Docket Nos. 50-254, and 50-265

Reference: August 1995 Quad Cities Units 1 and 2 Corner Room Structural Steel operability evaluation, including calculation No. QDC-0020-S-0055.

During our March 28, 1996 telephone conference call you requested a formal response to the following six questions regarding the operability evaluation of the Quad Cities corner room structural steel. As noted during this call, we are providing a response to five of the six questions at this time. Our response to question 5 will be submitted by Friday, April 5, 1996.

Question 1) Provide the Basis for the 10% overstress factor used in performing the operability evaluation.

In the development of qualifications for the operability of the framing steel in the corner rooms, the following justification is applied for the acceptance of calculated stresses in excess of the American Institute of Steel Construction (AISC) code allowables. For additional support in the justification of these allowables refer to the attached general position paper regarding the Justification of Increased Allowables Above Code Values (Attachment No. 1).

1.0 Response Spectra Conservatism

Quad Cities response spectra has been conservatively developed based upon the following factors which have been included in the design basis of the plant.

a) The current seismic analysis is based upon a conservative application of the Golden Gate time history. In addition, during the development of Spectra which were used for the design of structures, all modes up to 33 cps were included. In the original spectra which formed the initial licensing basis for the plant, only the Golden Gate time history

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and modes up to 8 cps were included. The resulting conservative spectra have been demonstrated to be as much as three times larger than the original design basis spectra for response on the rigid side of the peak. Since most structural systems are in this zone the current spectra are conservative.

b) The analytical simplifications used include conservative scaling of the input time history, use of 2% structural damping versus 7% currently allowed by the regulatory guides, and application of the input ground motion at the base of the structure.

2.0 Application of Loading

The loading applied to the computer model of the framing includes a distributed uniform floor loading of 50 psf to account for miscellaneous small attachments and personnel. This loading is excessive given the controlled nature of the corner rooms and the limited access allowed during operation. This number could be reduced significantly based upon the actual conditions in the area.

Piping analysis loads were conservatively based upon a series of isolated sub-system analyses and the resulting peak loads were applied simultaneously to the framing. The results of these analyses were combined using linear addition (absolute sum). These loads could be developed in a combined model of the piping system, heat exchanger and support steel and the results applied in accordance with the actual coupled behavior, thereby reducing the impact of the loading significantly on the critical framing. Results on similar analyses of coupled versus uncoupled models have shown reductions of up to 50% in the seismic loading applied to the structure.

For the purposes of the operability assessment, the damping used in the model could be increased to the values currently applied in the Reg. Guides. The current basis for damping in the seismic calculations is 0.5% damping for the piping and 2% damping for the structure. Since the majority of this framing is bolted, 7% damping could be justified for the structure.

3.0 Acceptance Criteria

In embedded plate assessments on the Quad Cities plant, the use of a strain based analytical approach was approved by the NRC. The effect of using the strain based process would increase the effective allowable stress in the members far greater than the 10% increased interaction being applied for the beams and connections. Also, the plant design basis includes the use of plastic design processes in the design of steel framing.

Increased material strength beyond the minimum specified yield strength in A36 steel have been seen in the results of testing performed for several other facilities (see Attachment 1, section 5). General industry work in reviewing the actual material strength has identified a 10% to 20% increase in the yield stress of A36 steel.

4.0 Conclusion

The affects of these three items provides the plant specific substantiation for the generic justification provided in Attachment 1. The cumulative effect of the identified conservatism would far exceed a 10% increase in allowable stresses, and thus is acceptable for use in reassessment of the Quad Cities Corner Room Structural Steel.

Question 2) Provide the criteria and supporting basis used to select the beams and connections for the bounding cases. Include a summary of the walkdown information used to support this assessment.

To establish a high level of confidence that the corner room structural steel as currently installed will remain operable under the UFSAR defined Safe Shutdown Earthquake (SSE) loading condition, the Interaction Coefficients (ICs) for the beams and connections from the preliminary Load Monitoring System (LMS) analyses of the four corner rooms were reviewed. The selection of the critical items to be evaluated for the operability was made based on the following criteria :

1. Review of the relative magnitude of ICs for beams and connections.

2. Identify items with ICs greater than 1.1 for further screening.

3. Identify items with reinforcements which are installed but not reflected in the LMS analysis. These reinforcements were provided after the initial LMS analysis was performed, but prior to this operability assessment.

4. Grouped items with ICs greater than 1.1 based on the similarity of the physical configuration. Beams were grouped based on member sizes and load orientation. Connections were grouped based on connection configuration, connecting element (building structure or framing into other beam), critical connection element (e.g. angles, bolts, beam web).

Based on the above criteria and the beam and connection ICs as tabulated on tables 2.1 through 2.4 of Attachment 2, the following critical members and connections were selected.

TABLE 1 - SUMMARY OF CRITICAL BEAMS AND CONNECTIONS

Room	Beam No.	Left End Conn.	Right End Conn.	Remarks
Q1SE	B1	N/A	N/A	Beam Check only
Q1SE	B4	N/A	N/A	Beam Check only
Q1SE	B8	N/A	N/A	Beam Check only
Q2SE	B8	YES	YES	Conn. Check only
Q2SE	B10	N/A	YES	Conn. Check only

SUMMARY OF PIPE SUPPORT ATTACHMENTS TO THE CORNER ROOM STEEL

Listed below is a summary of the number of pipe supports attached to the structural steel framing in the Residual Heat Removal (RHR) corner rooms. These walkdowns were performed in 1991. The preliminary LMS analysis results as indicated in Tables 2.1 through 2.4 of Attachment 2 include the loads from these pipe support attachments to the steel framing.

Room	Number of supports
Q1NE	20
Q1SE	19
Q2NE	12
Q2SE	11

Question 3) Did the preliminary analyses performed to support the operability evaluation include the weight of the lead blankets that are attached to the heat exchangers?

The weight of the lead shielding blankets was included in the preliminary analysis that was used as the basis for the operability assessment.

Question 4) **Provide the basis for the criteria defined on page 4 (item 2) and page 5 (item 2, part 1 and items 2 and 3, part 2) of calculation QDC-0020-S-0055.**

a) Provide the technical basis for reducing the torsional stresses in the structural steel by accounting for the actual rotational stiffnesses of the pipe support auxiliary steel as well as the structural steel.

The preliminary LMS analysis included the pipe hanger reactions on the structural steel framing based on the original Report of Loads (ROLs). The reactions included high torsional moments on the structural beams. This caused most of the very high beam and connection interaction coefficients as shown on Attachment 2. The torsional moments from auxiliary steel is the result of assuming the auxiliary steel to be fixed at the supporting structural steel. As part of the reconciliation effort and to help reduce the beam and connection overstresses, the torsional moments were virtually eliminated by accounting for the appropriate relative stiffness of the auxiliary steel and the structural steel. The revised support reactions were calculated using refined analysis techniques.

b) Provide a more detailed description of the pseudo stability loads used in the preliminary analyses, and clarify why it is conservative to combine these loads with seismic.

In determining the structural beams allowable bending stress (Fbx), the secondary beams were considered to provide lateral torsional restraints for the supporting beams where necessary to reduce the beam's unbraced length. As a result, the pseudo stability loads were calculated and conservatively combined in the LMS analysis in addition to the seismic loads. This is not required per the UFSAR nor applicable codes and thus can be eliminated from the seismic combinations. This stability load check will now be performed separately from the seismic load combinations.

c) Provide a more detailed explanation of the technical basis for the 10% overstress criteria (i.e. in place material strength, in phase application of seismic reversible loads and use of maximum service level loads).

See the response to question 1 for a detailed response addressing all of these topics.

Question 5) **For the one beam and one connection that have an interaction coefficient that is greater than or equal to 1.10, provide the revised analyses or justification to demonstrate compliance with your stated acceptance criteria.**

The response to this question will be submitted on or before April 5, 1996.

Question 6) After the reinforcements of the Unit 1 steel are completed, can the Unit 2 reinforcements be performed safely with the unit on line? If they can't be performed while the unit is on line, when can they be done?

The design work for Unit 2 will proceed after the completion of the installation of the Unit 1 corner room steel reinforcements. Installation on line would require voluntary entry into an Limiting Condition for Operation (LCO). The risk associated with this and interactions with other systems must be further evaluated. The Unit 2 corner room steel reinforcements will be installed in accordance with the guidance provided in Generic Letter 91-18, that is, prior to completing the next scheduled refueling outage or any other unplanned outage that is of sufficient duration to allow the completion of the reinforcement.

This completes the first submittal of our response to your request for additional information on March 28, 1996. As agreed during our telephone conference we will provide a response to the remaining question by April 5, 1996.

To the best of my knowledge and belief, the statements contained in this document are true and correct. In some respects these statements are not based on my personal knowledge, but on information furnished by other Commonwealth Edison employees, contractor employees, and/or consultants. Such information has been reviewed in accordance with company practice, and I believe it to be reliable.

If there are any questions concerning this matter, or need for further clarification, please contact this office.

Sincerely,

ES Kraft, Jr.

E. S. Kraft, Jr.
Site Vice President

Attachments: 1. Sargent & Lundy Structural Design Standard E5.0 Support for Increases in Allowable Stresses Above Code Defined Limits

2. Summary of Beam and Connection Interaction Coefficients

cc: H. J. Miller, Regional Administrator - RIII
R. M. Pulsifer, Project Manager - NRR
C. G. Miller, Senior Resident Inspector - Quad Cities

Attachment 1

Sargent & Lundy Structural Design Standard E5.0
Support for Increases in Allowable Stress
Above Code Defined Limits

1.0 Purpose

The purpose of this report is to provide an acceptable level of increase in allowable stresses for existing connections, beams and structural steel elements of power stations. This increase in allowable stresses can be justified on a generic basis to allow relief in the qualification effort for members which have been shown to be slightly above the code allowable conditions. The acceptance limit of 1.10 of the code allowable stresses is addressed in S&L Standard E5.0 provided proper substantiation exists. The scope of this paper is to provide the engineer with a series of concepts and areas to consider in reviewing the basis for acceptance of limited stress increases above code allowables. The user of these concepts and ideas is responsible for documentation and support of these on the individual project without violating any project or plant specific design basis.

2.0 Scope

The scope of this report is to assess the conservatisms which are contained in the development of loading, the methods of loading combinations, material strength parameters and general analytical procedures. No attempt will be made to assess the application of any of these factors into a specific design. The factors involved will be based upon either documented testing programs, established industry codes or standards, or published documents. The focus of this development will be on nuclear power stations. Many of the factors discussed may also apply to fossil power stations. It is intended that any increase in allowable stresses will be limited to the evaluation of installed members where modification is difficult or not practical. The use of increased allowables should be avoided for new designs or modifications to existing structures.

3.0 Development of Loading

Live Loads

The development of live loads for the structural framing in power plant structures is generally based upon the heaviest component which will be applied to the floor during the life of the station. This load is most likely to be applied during shut down of the power station, or during equipment removal and servicing. During the operation of the power station few personnel are active and generally little or no major maintenance activities in the plant occur. While true in general access areas of the station, this can be particularly true in radiation controlled areas of nuclear stations. In actual practice this live loading is often reduced to a minimum level, such as 50 psf which corresponds to the Uniform Building Code (UBC) value for general office areas. In practical terms, a 50 psf load is equal to 2-250 lb. men standing in an area slightly larger than 3 feet square. Even this loading is quite conservative when considering an entire floor. Further support for reduction of live loading can be found in the Uniform Building Code, which allows for live load reductions to be applied when the tributary areas of the structural member exceed various levels. This applies to general commercial buildings where the normal condition of the structure is for it to be occupied. This logic applies only to distributed floor loads and not to loading induced by equipment. The use of conservative estimates of live loads results in an increase in maximum stresses of approximately 2% to 5% and can be as high as 20% for some structures.

Seismic Loads

The current requirements for the determination of seismic loading for nuclear power stations are based upon the procedures identified in NUREG-0800. The specific sections of the SRP that address the seismic analysis of structures are Sections 3.7.1, 3.7.2, 3.7.3. In the development of the seismic forces for a structure, a number of conservative assumptions may have been made concerning the magnitude of the input ground motion, the soil structure interaction, structural damping of the model, simplifications in the structural modeling, combination of modal responses, etc. The American Society of Civil Engineers (ASCE) publication "Uncertainty and Conservatism in Seismic Analysis and Design of Nuclear Facilities", 1986, Table 9.1 presents some ranges in the conservatism inherent in the structure analysis and design. The response of a structure is predicted to have a factor of safety between 1.6 and 3.2 based upon the general level of conservatism built into the analytical process. When consideration is given to the structural capacity, the total factor of safety rises to between 4 and 19. This is independent of and in addition to any site related conservatisms. If only the lower value was used, it can be seen that there is at least 60% additional capacity available if a more refined analysis is performed.

In the SRP, the cut-off for all analysis is the point where a change in the analysis will result in a 10% or less effect on the analytical result. From these documents it can be seen that there are some conservative assumptions made in the analytical basis for the development of the response spectra used in the plant. The design basis of the plant should be reviewed to determine the quantity of this conservatism which can be justified in any given analysis. If the plant design basis applies simplified methods such as the UBC simplified approach these reductions will not be possible.

When loading from multiple sub-systems is developed and applied to structures for the sizing of the structural members, the peak response of the sub-systems is seldom, if ever, at the same frequency. If these responses are linearly added together on the structure, significant conservatism can be introduced into the structures design. It is possible to prepare coupled models which provide a more realistic response of the sub-systems response on the structure. Another way of achieving this reduction is to perform a more detailed combination of the stresses from each load using an Square Root Sum of the Squares (SRSS) summation of the stress resulting from frequencies which are non-concurrent. These types of approaches have been approved by the NRC in various design basis documents. We have seen through the use of these coupled models or refined combinations of load components that the resulting loading applied to the structure can be reduce by 20% to 50%.

4.0 Allowable Stresses

SRP Section 3.8.4 provides guidance regarding the loading combinations for structures and components other than containment buildings. In these combinations the use of allowable stresses above the AISC or American Concrete Institute (ACI) allowable stresses is allowed in cases where abnormal or faulted loads are applied. The stress limits in several of these cases exceeds the published yield stress of the materials. For example, in load case 3.c.ii.a.1 of the section 3.8.4 the allowable stress level is 1.6 S. In cases of bending for compact sections the major axis bending stress is allowed at 0.66 Fy and the minor axis bending is 0.75 Fy. Using the SRP acceptance basis specified, the allowable major axis bending stress in members would be 1.056 Fy and the minor axis bending would be 1.2 Fy. S&L has conservatively limited the stress increases to less than 0.95 Fy in all cases. The difference between these values is 0.106 Fy for major axis and 0.25 Fy for minor axis bending. Where plastic design is used the comparable load cases are allowed to go the full plastic allowable under faulted load cases (1.0 Mp). S&L's normal practice is to limit the allowable stress to less than 0.95 Mp when using plastic allowables. This is effectively a 0.05 Mp increase in allowable stresses that is not included in the S&L design methodology. Thus, under plant design conditions using the faulted loads, a minimum increase of 10% with elastic allowables and 5% with plastic allowable can be permitted.

Another significant attribute is consideration of the primary stresses versus secondary stresses. Secondary stresses are caused where local yielding of a structural element occurs allowing redistribution of forces to another portion of the structure. An example of these secondary stresses is in a clip angle connection where rotation will develop in a simple framing connection. If stresses were determined for these connections, the connection elements would appear to be highly stressed. After the connection rotation occurs the member will tend to resolve the loading through increased deflection and the resulting system remains stable. AISC recognizes this redistribution of forces as long as stability and serviceability of the framing is not affected. Primary stresses in members and conditions where the stresses in a member or connection cannot redistribute the load without failure of the member. An obvious example of this condition would occur at the connection of a cantilever beam subjected to axial load and bending. Any attempt to redistribute the loading would cause the failure of the member through excessive deflection or collapse. It is important that in evaluating the conditions of secondary stresses, for those conditions where redistribution of forces is possible, to verify that the elements are not governed by elastic allowable limits specified in the AISC code. In these cases, the code allowables may be increased.

The allowable stresses developed in building codes are established with knowledge of the tolerances allowed in the fabrication and construction of the structural element. The intent of the tolerances is to allow some variation to occur in the construction without jeopardizing the load carrying capacity of the members. If a member is fabricated, installed and inspected to a more stringent acceptance criteria, these construction variables are no longer a concern, provided the inspections are done to assure that the final installed dimensions are included in the analysis. Due to the highly controlled inspection and quality control process in place at most nuclear power stations these construction errors are severely limited and thus the normal code allowables stresses can be increased.

5.0 Material Strength

Over time S&L has had occasion to review the material certifications from actual framing and connections installed in numerous nuclear power stations. In addition, for several special cases actual coupons of the material were taken from beams where large loading increases would have required replacement without larger allowable stresses. In each of these cases, the actual yield strength of the material has been found to exceed the specified design limit. Material coupons taken from the Dresden Station drywell framing (for A36 steel) had a yield strength between 41.895 ksi and 43.780 ksi. Similar coupons taken from A36 steel of 1960 vintage fossil plants showed similar results. Reviews performed on the Byron Station material certification records show actual material yield stresses within this same range. Since consistent results were obtained from several sources, it can be seen that the material composition of A36 steel was consistent over a long period of time (approximately 20 years). This would justify the use of a value higher than the normal code allowable stresses. These results are supported by "Properties of Steel for Use in LRFD" published in the ASCE Journal of the Structural Division. In this publication, Table 2 records the test results for over 400 tests in the United States and the Mean Mill Fy which resulted was 44 ksi. This is consistent with the information from our historical data. Even when samples where taken from the most critical portion of the section of a shape the result was mean Fy = 41.3 ksi. Similar work was done for piping and valves and NUREG CR2137 found the steel strength to be about 10% to 20% above the published values. This data confirms the conservative nature of the yield stress for the material and supports the use of higher allowables stresses for steel framing.

6.0 Conclusion

In conclusion, it can be seen that there are a number of factors which can contribute to conservatism in the development of loading, combination of loads, analysis of structures and the material allowable stresses. All of these factors will not apply to all structural elements. Many of these conservative approaches can result in apparent increases in stresses of 20% or more. Summarized below in Table 6.1 is a listing of the various sources of conservatisms that were evaluated. Since many of these values are difficult to quantify in a specific case, therefore, it is required to limit the increase in allowable stress interaction to a total of 10%. If additional increases in capacity are required the engineer should prepare additional refined analysis which reduce or eliminate the need for further increases in stresses greater than 10 percent. If the required increase in code allowables is less than 10%, the engineer should verify and document that those conservatisms exist.

In support of this position, S&L has surveyed other major architectural/engineering firms and has found that they also use a similar 10% increase in allowable stresses in the justification of installed members. These increases in allowable stresses have been used in commercial structures, fossil plants and nuclear plants throughout the United States.

Table 6.1 Summary of Sources of Conservatism

Source of Conservatism (1)	Estimated Magnitudes	Remarks
Live Loads	2 - 5 %	
Seismic Loads (2)	20 - 50 %	Based on representative experience
SRP/Code Allowables	5 - 10 %	Increases above the SDS-E5.0 stress limits
Actual Material Yield Strength	10 - 20 %	Applicable for A-36 steel procured and fabricated using approved suppliers

(1) This summary does not include a quantitative estimate of the conservatism associated with such items as secondary stress redistribution and the result of rigorous inspection.

(2) Subject to plant specific seismic model characteristics.

7.0 References

1. Uncertainty and Conservatism in the Seismic Analysis and Design of Nuclear Facilities, ASCE 1986.

2. NUREG 0800 - Standard Review Plan.

3. Properties of Steel for Use in LRFD, Journal of the Structural Division, ASCE, 1978, T. V. Galambos and M. K. Ravindra.

4. AISC Steel Construction Manual.

5. AWS D1.1 Structural Welding Code.

6. Uniform Building Code, 1994 ed. international Conference of Building Officials

7. S&L Standard E[illegible] and Back-up Calculations

SUMMARY OF BEAM AND CONNECTION ICs

Table 2.1 : UNIT 1 NORTHEAST CORNER ROOM

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
1 16WF36	BEAM BEND.	2.16	
	LEFT CONN.		
	ANGLE-BEND	1.40	
	BEAM-WEB	1.37	
	RIGHT CONN.		
	BEAM-WEB	10.58	See Note 1 on page 2
	CHEEK-PL	14.47	See Note 1 on page 2
	EMB-PL	2.51	
	ILEG-WELD	1.55	
	OLEG-WELD	1.75	
2 10WF33	RIGHT CONN.		
	ANGLE-BEND	10.17	See Note 1 on page 2
	BEAM-WEB	10.56	See Note 1 on page 2
	ANGLE-IL-FU	1.27	
	SS-WEB-FU	1.77	
	SS-WEB-FY	1.32	
3 10WF33	LEFT CONN.		
	ANGLE-BEND	2.07	
	RIGHT CONN.		
	ANGLE-BEND	5.40	See Note 1 on page 2
	BEAM-WEB	4.64	See Note 1 on page 2
4 24WF76 W/ WT REINF.	BEAM BEND.	1.95	
	LEFT CONN.		
	ANGLE-BEND	1.17	
	EMB-PL	2.65	
5 12WF53	LEFT CONN.		
	ANGLE-BEND	1.26	
	BEAM-WEB	1.67	
	RIGHT CONN.		
	BEAM-WEB	4.40	
	CHEEK-PL	5.40	
	EMB-PL	2.60	
	ILEG-WELD	1.81	
	OLEG-WELD	2.06	

SUMMARY OF BEAM AND CONNECTION ICs

TABLE 2.1 : UNIT 1 NORTHEAST CORNER ROOM CONT.

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
6 12WF53	BEAM BEND.	1.43	
	LEFT CONN.		
	ANGLE-BEND	1.19	
	BEAM-WEB	1.77	
	SS-WEB-FU	1.26	
	RIGHT CONN.		
	ANGLE-BEND	2.20	
	BEAM-WEB	2.66	
	SS-WEB-BEND	1.13	
8 14WF68	BEAM BEND.	1.77	
	LEFT CONN.		
	ANGLE-BEND	5.48	See Note 1 on page 2
	BEAM-WEB	3.90	See Note 1 on page 2
	SH-T-OL-BOLT	1.36	
	SS-ANG-IL-FU	1.25	
	SS-WEB-FU	1.55	
	RIGHT CONN.		
	ANGLE-BEND	4.69	See Note 1 on page 2
	BEAM-WEB	4.63	See Note 1 on page 2
	BLK-SHR-WEB	1.24	
	SF-BOLT-IL	1.28	
	SH-T-OL-BOLT	1.58	
	SS-ANG-IL-FU	1.44	
	SS-ANG-IL-FY	1.17	
	SS-WEB-FU	1.94	
	SS-WEB-FY	1.25	
10 21WF55	LEFT CONN.		
	BEAM-WEB	1.97	
	CHEEK-PL	1.26	
	EMB-PL	2.44	
15 2L6X4X.5	LEFT CONN.		
	EMB-PL	2.92	

Notes:

1. Significant amount or the majority of the overstress is due to the torsional moments on the beam from the auxiliary steel of pipe supports attachments. The torsional moments have been virtually eliminated based on refined analysis of the pipe support.

2. IC is without reinforcements which were installed but not reflected in the LMS analysis. These reinforcements were provided after the initial LMS analysis but prior to the operability evaluation.

SUMMARY OF BEAM AND CONNECTION ICs

TABLE 2.2 : UNIT 1 SOUTHEAST CORNER ROOM

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
1 16WF36	BEAM BEND.	2.22	***THIS BEAM WAS SELECTED FOR THE OPERABILITY EVALUATION***
	LEFT CONN.		
	ANGLE-BEND	1.56	
	BEAM-WEB	1.39	
	RIGHT CONN.		
	BEAM-WEB	8.15	See Note 1 on page 2
	CHEEK-PL	8.97	See Note 1 on page 2
	EMB-PL	2.29	
	ILEG-WELD	1.42	
	OLEG-WELD	1.61	
2 10WF33	RIGHT CONN.		
	ANGLE-BEND	8.64	See Note 1 on page 2
	BEAM-WEB	8.50	See Note 1 on page 2
	SS-IL-FU	1.11	
	SS-WEB-FU	1.58	
	SS-WEB-FY	1.19	
3 10WF33	LEFT CONN.		
	ANGLE-BEND	1.44	
	RIGHT CONN.		
	ANGLE-BEND	7.92	See Note 1 on page 2
	BEAM-WEB	7.10	See Note 1 on page 2
	SS-WEB-FU	1.28	See Note 1 on page 2
4 24WF76	BEAM BEND.	2.09	***THIS BEAM WAS SELECTED FOR THE OPERABILITY EVALUATION***
	LEFT CONN.		
	ANGLE-BEND	1.62	
	WEB-BEND	1.23	
	EMB-PL	3.02	
5 12WF53	LEFT CONN.		
	ANGLE-BEND	1.79	
	BEAM-WEB	2.21	
	RIGHT CONN.		
	BEAM-WEB	4.75	
	CHEEK-PL	5.58	
	EMB-PL	2.80	
	ILEG-WELD	1.99	
	OLEG-WELD	2.25	

SUMMARY OF BEAM AND CONNECTION ICs

TABLE 2.2 : UNIT 1 SOUTHEAST CORNER ROOM CONT.

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
6 12WF53	BEAM BEND.	1.33	
	LEFT CONN.		
	ANGLE-BEND	3.46	See Note 1 on page 2
	BEAM-WEB	3.82	See Note 1 on page 2
	SS-WEB-FU	1.46	
	RIGHT CONN.		
	ANGLE-BEND	1.46	See Note 1 on page 2
	BEAM-WEB	1.93	See Note 1 on page 2
8 14WF68	BEAM BEND.	1.64	***THIS BEAM WAS SELECTED FOR THE OPERABILITY EVALUATION***
	LEFT CONN.		
	ANGLE-BEND	1.94	
	BEAM-WEB	2.03	
	BLK-SHR-WEB	1.23	
	SF-BOLT-ILEG	1.28	
	SH-T-OL-BOLT	1.19	
	SS-ANG-IL-FU	1.25	
	SS-WEB-FU	1.55	
	RIGHT CONN.		
	ANGLE-BEND	2.21	
	SF-BOLT-IL	1.13	
10 21WF55	LEFT CONN.		
	EMB-PL	1.24	
	LEFT CONN.		
	BEAM-WEB	2.02	
	CHEEK-PL	1.30	
	EMB-PL	1.75	
15 2L6X4X.5	LEFT CONN.		
	EMB-PL	3.76	

SUMMARY OF BEAM AND CONNECTION ICs

TABLE 2.3 : UNIT 2 NORTHEAST CORNER ROOM

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
1 16WF36	BEAM BEND.	6.88	Beam IC does not include beam reinforcement. The beam has been reinforced with side plates after the LMS analyses was performed.
	RIGHT CONN.		
	BEAM-WEB	5.17	See Note 1 on page 2
	CHEEK-PL	5.50	See Note 1 on page 2
	EMB-PL	1.40	
	OLEG-WELD	1.24	
2 10WF33	RIGHT CONN.		
	BEAM-WEB	1.14	
3 10WF33	BEAM BEND.	1.41	See Note 1 on page 2
	LEFT CONN.		
	ANGLE-BEND	2.08	
	RIGHT CONN.		
	ANGLE-BEND	15.40	See Note 1 on page 2
	BEAM-WEB	15.05	See Note 1 on page 2
	SH-T-OL-BOLT	2.21	See Note 1 on page 2
	SS-ANG-IL-FU	2.82	See Note 1 on page 2
	SS-WEB-FU	3.84	See Note 1 on page 2
	SS-WEB-FY	2.90	See Note 1 on page 2
4 24WF76 W/POST	BEAM BEND.	3.02	Beam IC does not include the existing post which was added to reinforce the beam after the LMS analyses was performed.
	LEFT CONN.		
	ANGLE-BEND	1.19	See Above
	EMB-PL	2.28	See Above
5 12WF53	RIGHT CONN.		
	BEAM-WEB	11.3	See Note 2 on page 2
	CHEEK-PL	15.80	See Note 2 on page 2
	EMB-PL	4.92	See Note 2 on page 2
	ILEG-WELD	2.19	See Note 2 on page 2
	OLEG-WELD	2.43	See Note 2 on page 2
6 12WF53	BEAM BEND	1.47	
	LEFT CONN.		
	ANGLE-BEND	4.21	See Note 1 on page 2
	BEAM-WEB	4.21	See Note 1 on page 2
	SS-WEB-FU	1.43	See Note 1 on page 2
	RIGHT CONN.		
	ANGLE-BEND	1.82	See Note 1 on page 2
	BEAM-WEB	1.92	See Note 1 on page 2

SUMMARY OF BEAM AND CONNECTION ICs

TABLE 2.3 : UNIT 2 NORTHEAST CORNER ROOM CONT.

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
8 14WF68	BEAM BEND.	1.56	
	RIGHT CONN.		
	ANGLE-BEND	1.47	
	BEAM-WEB	1.87	
	ILEG-BOLT	1.21	
	OLEG-BOLT	1.17	
	SS-WEB-FU	1.56	
9 12WF53	LEFT CONN.		
	ANGLE-BEND	3.12	
	BEAM-WEB	1.51	
10 21WF55	LEFT CONN.		
	BEAM-WEB	1.72	
	CHEEK-PL	1.36	
	EMB-PL	1.79	

SUMMARY OF BEAM AND CONNECTION ICs

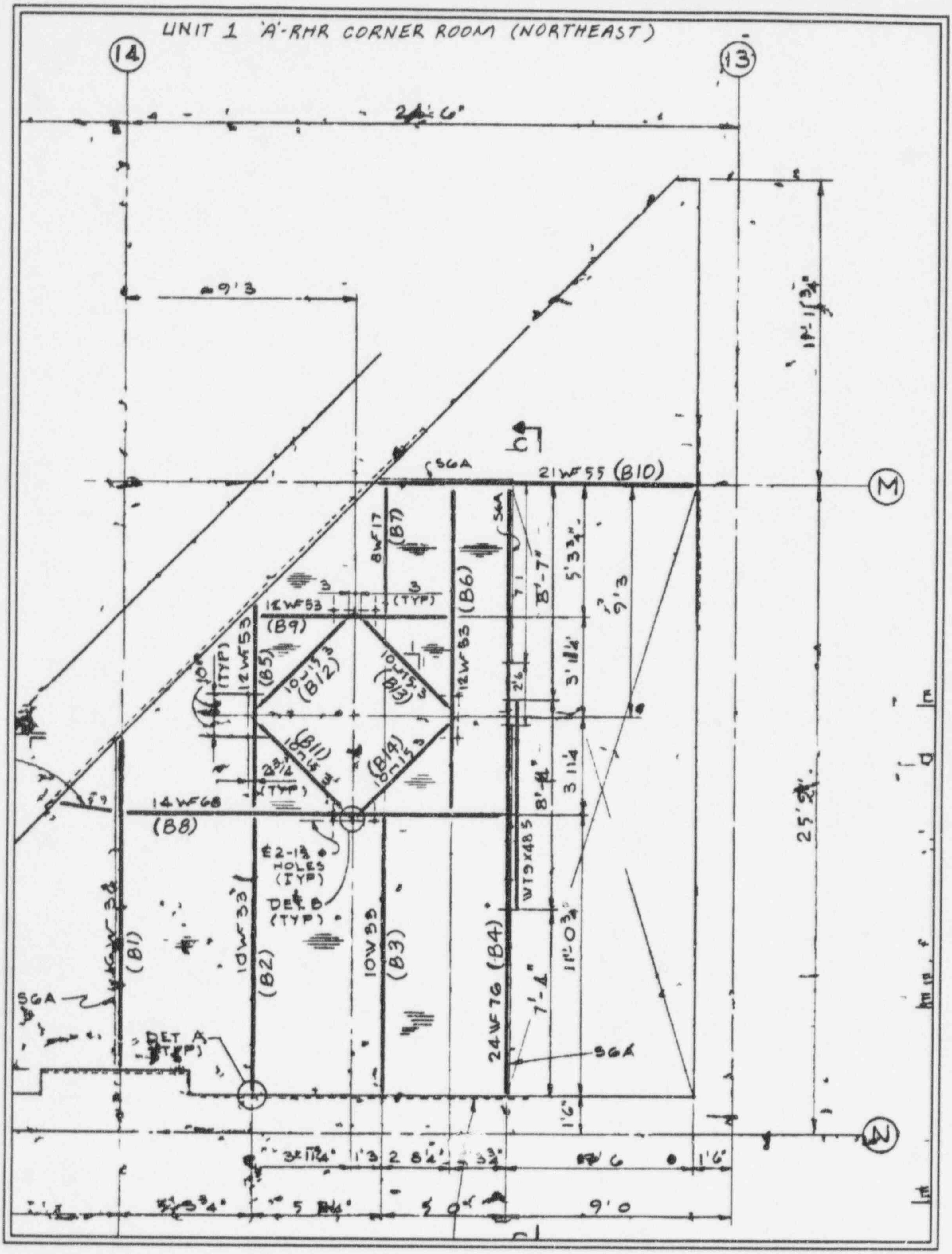
TABLE 2.4 : UNIT 2 SOUTHEAST CORNER ROOM

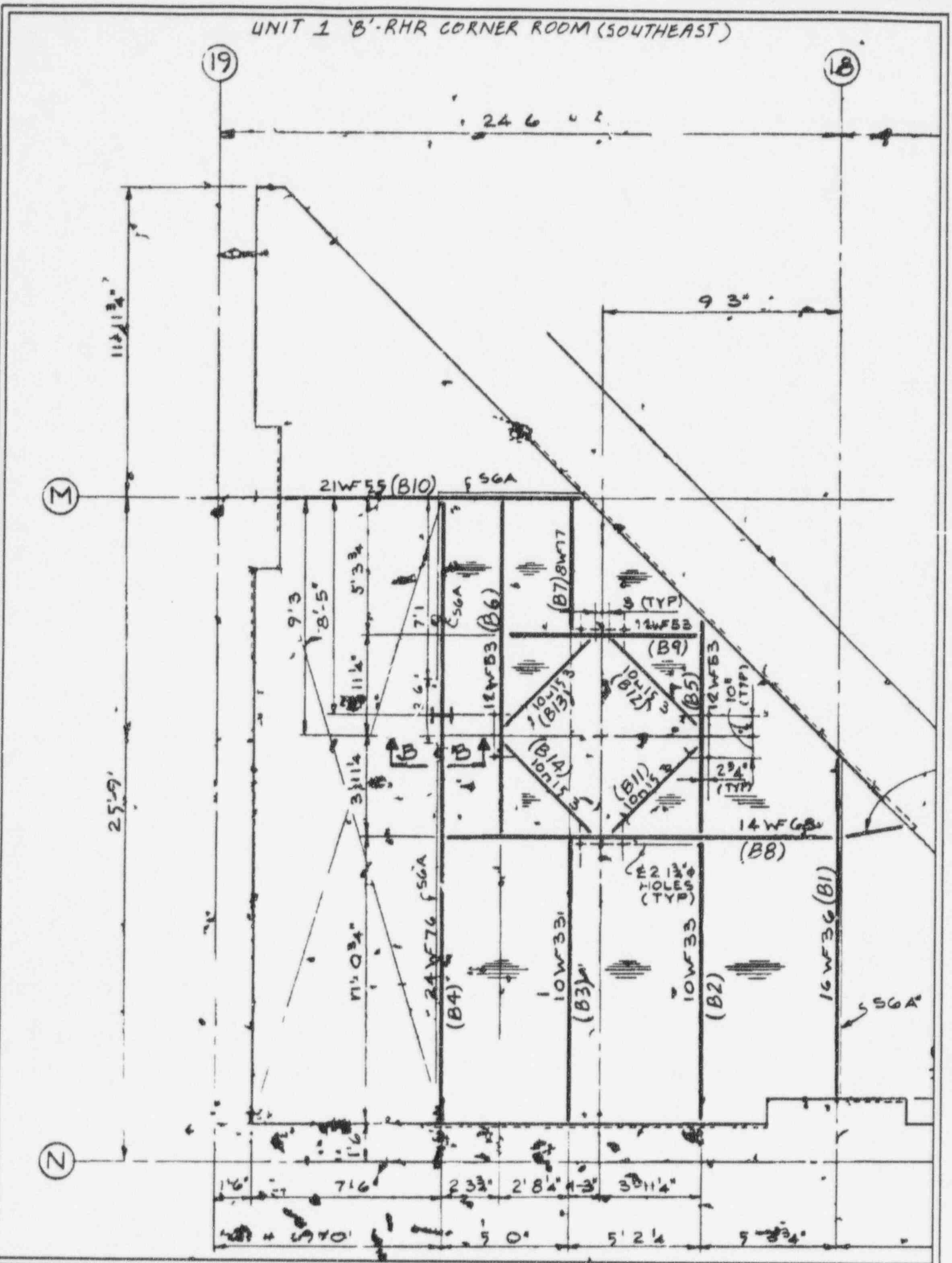
BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
1 16WF36	RIGHT CONN.		
	BEAM-WEB	5.13	See Note 1 on page 2
	CHEEK-PL	5.79	See Note 1 on page 2
	EMB-PL	1.56	
	ILEG-WELD	1.18	
	OLEG-WELD	1.35	
2 10WF33	RIGHT CONN.		
	ANGLE-BEND	2.99	See Note 1 on page 2
	BEAM-WEB	2.61	See Note 1 on page 2
3 10WF33	LEFT CONN.		
	ANGLE-BEND	2.54	
	RIGHT CONN.		
	ANGLE-BEND	3.70	See Note 1 on page 2
	BEAM-WEB	2.62	See Note 1 on page 2
B4 24WF76	LEFT CONN.		
	EMB-PL	2.24	
5 12WF53	LEFT CONN.		
	ANGLE-BEND	1.23	See Note 1 on page 2
	BEAM-WEB	1.76	See Note 1 on page 2
	RIGHT CONN.		
	BEAM-WEB	5.03	
	CHEEK-PL	5.22	
	EMB-PL	2.38	
	ILEG-WELD	1.84	
	OLEG-WELD	2.06	
6 12WF53	BEAM BEND.	1.40	
	LEFT CONN.		
	ANGLE-BEND	1.83	See Note 1 on page 2
	BEAM-WEB	2.41	See Note 1 on page 2
	RIGHT CONN.		
	ANGLE-BEND	1.66	See Note 1 on page 2
	BEAM-WEB	1.95	See Note 1 on page 2

SUMMARY OF BEAM AND CONNECTION ICs

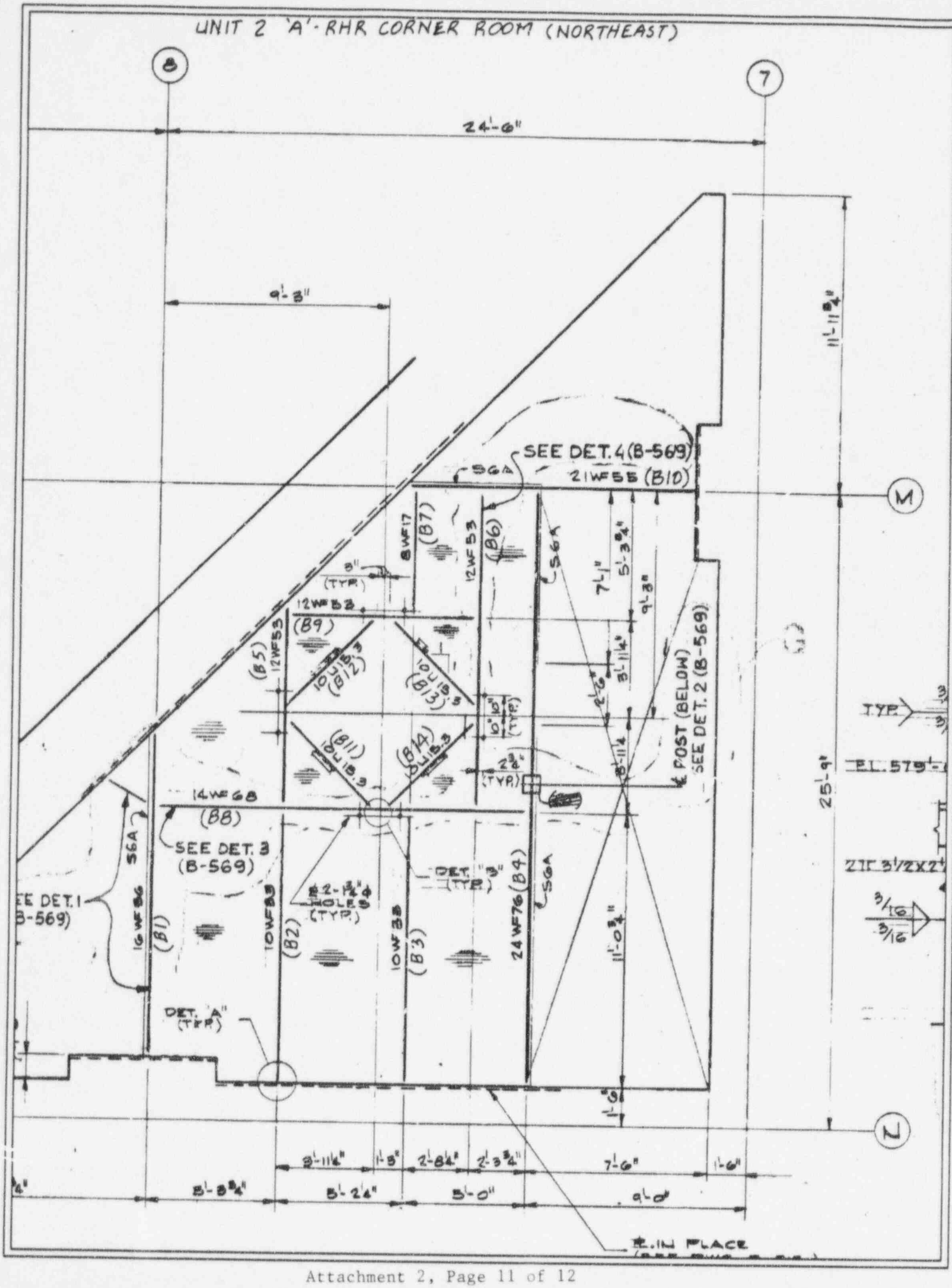
TABLE 2.4 : UNIT 2 SOUTHEAST CORNER ROOM CONT.

BEAM No. SIZE	COMPONENT	PRELIMINARY IC	REMARKS
8 14WF68	BEAM BEND.	1.41	
	LEFT CONN.		***THIS CONNECTION WAS SELECTED FOR THE OPERABILITY EVALUATION***
	ANGLE-BEND	1.16	
	BEAM-WEB	1.29	
	BLK-SHR-WEB	1.12	
	SF-BOLT-ILEG	1.16	
	SS-WEB-FU	1.33	
	RIGHT CONN.		***THIS CONNECTION WAS SELECTED FOR THE OPERABILITY EVALUATION***
	ANGLE-BEND	1.70	
9 12WF53	RIGHT CONN.		
	WEB-BEND	1.16	
10 21WF55	BEAM BEND.	1.23	
	LEFT CONN.		
	EMB-PL	1.36	
	RIGHT CONN.		***THIS CONNECTION WAS SELECTED FOR THE OPERABILITY EVALUATION***
	BEAM-WEB	3.40	See Note 1 on page 2
	CHEEK-PL	3.07	See Note 1 on page 2
	EMB-PL	2.20	
15 2L6X4X.5	BEAM BEND.	1.97	
	LEFT CONN.		
	EMB-PL	1.31	
	RIGHT CONN.		
	CEA-PL	2.05	

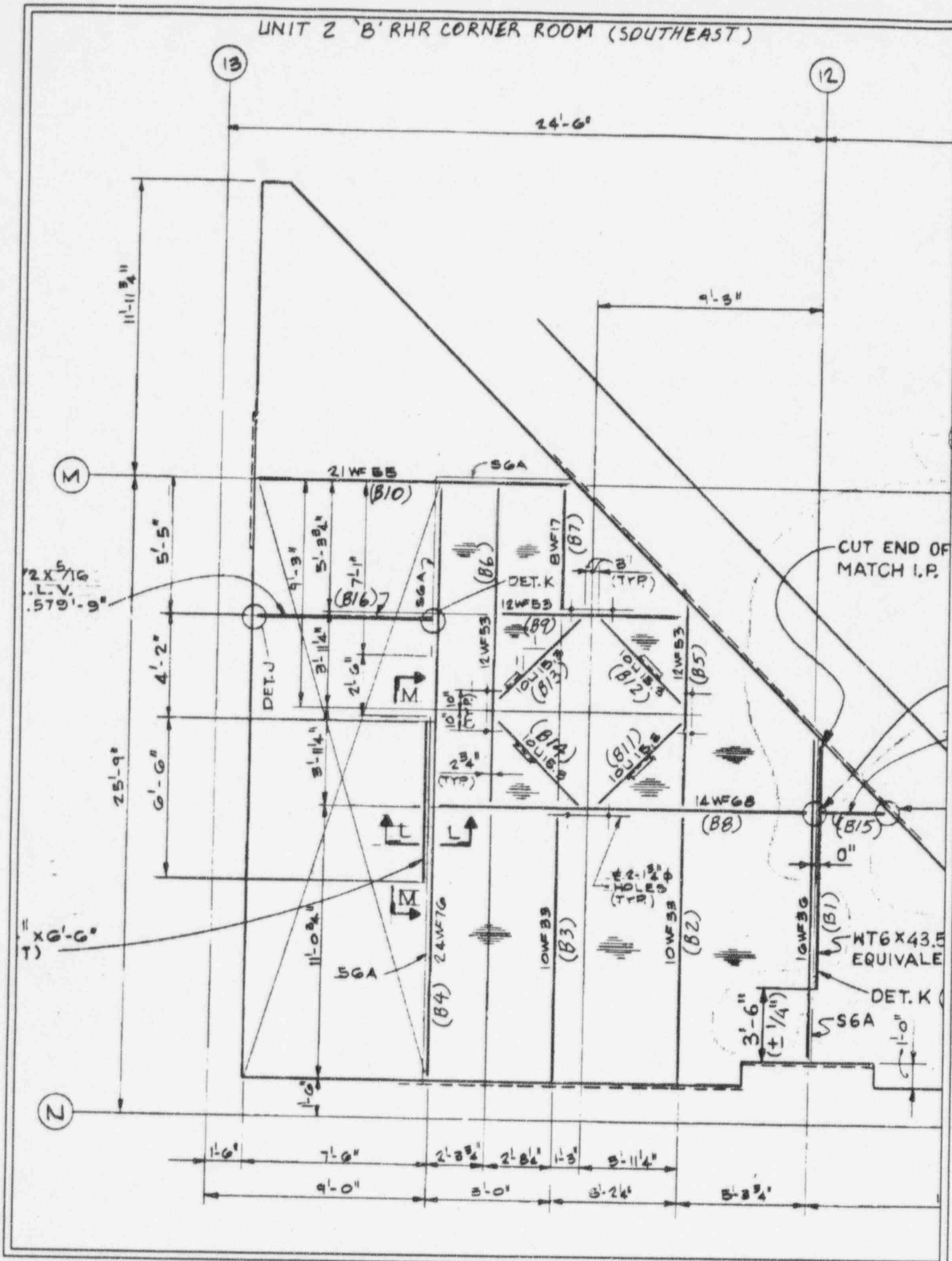

UNIT 1 'A'-RHR CORNER ROOM (NORTHEAST)
14
13
M
N
21WF55 (B10)
14WF68
(B8)
12WF53
(B9)
8WF17
(B7)
12WF53
(B6)
12WF53
(B5)
10W15.3
(B12)
(B13)
(B11)
(B14)
10W33
(B2)
10W33
(B3)
24WF76 (B4)
(B1)
WT9X48.5
SGA
DET A
DET B
(TYP)
HOLES
(TYP)
1'-6"
9'0
1'3
1'6"



UNIT 1 'B'-RHR CORNER ROOM (SOUTHEAST)
19
18
M
N
24 6
9 3"
21WF55 (B10)
(B7) 8W17
3 (TYP)
12WF53
(B9)
12WF53
(B6)
12WF53
(B5)
(B13)
(B12)
(B14)
(B11)
14 WF68
(B8)
24 WF76
(B4)
10WF33
(B3)
10WF33
(B2)
16 WF36 (B1)
HOLES
(TYP)
(TYP)
9'3
8'5"
5'3¾"
7'1
25'-9'
1'6"
7'6
2 3¾"
2' 8¼"
1-3"
5 0"
5' 2¼



UNIT 2 'A'-RHR CORNER ROOM (NORTHEAST)
8
7
M
N
24'-6"
9'-3"
11'-11 3/4"
25'-9"
SEE DET. 4 (B-569)
21WF55 (B10)
SGA
8WF17
(B7)
12WF53
(B6)
3" (TYP.)
12WF53
(B9)
(B5)
12WF53
10 [15.3 (B12)
10 [15.3 (B13)
(B11) 10 [15.3
(B14) 10 [15.3
10" 10" (TYP.)
2 3/4" (TYP.)
7'-1"
5'-3 3/4"
9'-3"
3'-11 1/4"
2'-6"
3'-11 1/4
℄ POST (BELOW) SEE DET. 2 (B-569)
14WF68
(B8)
SEE DET. 3 (B-569)
SEE DET. 1 (B-569)
16WF36 (B1)
10WF33 (B2)
10WF33 (B3)
24WF76 (B4)
2-1 1/4" ϕ HOLES (TYP.)
DET. "B" (TYP.)
DET. "A" (TYP.)
11'-0 3/4"
9"
1'
TYP.
3/16
EL. 579'
2 L 3 1/2 X 2
3'-11 1/4"
1'-5"
2'-8 1/4"
2'-3 3/4"
7'-6"
1'-6"
5'-3 3/4"
5'-2 1/4"
5'-0"
9'-0"
℞ IN PLACE


UNIT 2 'B' RHR CORNER ROOM (SOUTHEAST)
13
12
M
N
24'-6"
9'-3"
11'-11 3/4"
5'-5"
4'-2"
6'-6"
25'-9"
21WF55
(B10)
S6A
5'-3 3/4"
7'-1"
9'-3"
(B16)
8WF17
(B7)
(B6)
DET. K
8' (TYP)
12WF53
(B9)
DET. J
3'-11 1/4"
2'-6"
12WF53
10[15.3
(B13)
(B12)
(B5)
(B14)
(B11)
10" 10" (TYP)
2 3/4" (TYP)
14WF68
(B8)
(B15)
0"
CUT END OF MATCH I.P.
2-1 3/4" ϕ HOLES (TYP)
10WF33
(B3)
(B2)
16WF36
(B1)
WT6 X 43.5 EQUIVALE
DET. K
24WF76
(B4)
S6A
11'-0 3/4"
3'-6" (± 1/4")
1'-0"
1'-6"
7'-6"
2'-8 3/4"
2'-8 3/4"
1'-3"
5'-11 1/4"
9'-0"
5'-0"
3'-2 1/4"
5'-8 3/4"